PATERSON GROUP

Geotechnical Investigation

Proposed Residential Development

3996 Innes Road
Ottawa, Ontario

Prepared for Mr. Loutfi Frangian

Report PG5925-1 Revision 2 dated February 6, 2025

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Mr. Loutfi Frangian to conduct a geotechnical investigation for the proposed residential development to be located at 3996 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 the available drawings provided by the client, it is understood that a multi-story residential/commercial building is being considered for this site with one underground level.

Associated landscaped areas, paved driveways and access lanes are also anticipated. It is further anticipated that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on August 18, 2021 and consisted of advancing a total of 2 boreholes to a maximum depth of 6 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG5925-1 - 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. 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. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the auger, and split-spoon samples were recovered from the boreholes are shown as AU, and SS 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.

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 the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The test hole locations 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 borehole locations along with the ground surface elevation at each test hole location are presented on Drawing PG5925-1 - 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 1 shrinkage test, 2 grain size distribution and hydrometer analyses, and 2 Atterberg limits tests were completed on selected soil samples. The results are presented in Subsection 4.2 and on Grain Size Distribution and Hydrometer Testing, and Atterberg Limit's Results and Shrinkage Test Results sheets presented in Appendix 1.

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, one of which was collected from BH 2-21. 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 with a gentle downslope towards the rear side of the existing building. The subject site consists of two lots occupied by single-family residential dwellings with associated landscaped areas, fences, and driveways.

The site is bordered by Innes Road to the north, by a commercial plaza to the west and south end, and a church to the east.

4.2 Subsurface Profile

Overburden

Generally, the soil profile at the test hole locations consists of topsoil underlain by fill extending to depths ranging from 0.5 to 0.6 m. The fill was generally observed to consist of brown silty sand with trace to some clay and some topsoil.

A hard to very stiff, brown silty clay layer was encountered underlying the fill. In one of the test holes, the silty clay was observed to transition into a very dense glacial till between 2.29 to 2.84 m depth below existing grade. The glacial till layer consists of brown silty sand with gravel, cobbles and boulders with trace of clay. Practical refusal to augering on inferred bedrock was encountered in both boreholes at depths ranging from 2.4 to 2.9 m below existing grade.

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 interbedded limestone and dolostone of the Gull River formation, with an estimated overburden drift thickness of 2 to 3 m depth.

Atterberg Limit and Shrinkage Tests

Atterberg limits testing, as well as associated moisture content testing, were completed on the recovered silty clay samples at selected locations throughout the subject site. The results of the Atterberg limits are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1.

Table 1 - Atterberg Limits Results					
Sample	**Depth (m)**	**LL (%)**	**PL (%)**	**PI (%)**	**Classification**
BH 1-21 SS3	1.5 – 2.1	61	27	34	CH
BH 2-21 SS3	1.5 – 2.1	67	31	36	CH
Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index. CH: Inorganic Clay of High Plasticity MH: Inorganic Silt of High Plasticity					

The results of the moisture contest test are presented in Table 2 and on the Soil Profile and Test Data Sheet in Appendix 1.

The results of the shrinkage limit test indicate a shrinkage limit of 3.81 and a shrinkage ratio of 2.04.

Table 2 – Moisture Content Results			
Borehole	**Sample**	**Depth (m)**	**Water Content (%)**
BH 1-21	AU1	0.1– 0.6	11.60
BH 1-21	SS2	0.7 – 1.3	37.93
BH 1-21	SS4	2.3 – 2.9	11.7
BH 2-21	AU1	3.9 - 4.5	10.91
BH 2-21	SS2	0.7 – 1.3	25.59
BH 2-21	SS4	2.3 – 2.9	37.18

Grain Size Distribution and Hydrometer Testing

Grain size distribution (sieve and hydrometer analysis) was also completed on two (2) soil samples. The results of the grain size analysis are summarized in Table 3 and presented on the Grain-size Distribution and Hydrometer Testing Results sheets in Appendix 1.

Table 3 - Summary of Grain Size Distribution Analysis					
Test Hole	**Sample**	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**
BH 1-21	SS2	0.0	9.6	38.4	52.0
BH 2-21	SS2	0.0	7.2	32.8	60.0

4.3 Groundwater

Groundwater levels were measured on August 20,2021 within the installed polytube piezometers. The measured groundwater levels are presented in Table 4 below.

Table 4 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1B-21	89.73	2.83	86.90	August 20, 2021
BH 2-21	90.35	0	90.35	

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 could be influenced by surface water infiltrating the backfilled boreholes. Therefore, long-term groundwater levels can also be estimated based on the observed color, consistency and moisture levels of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected below the bedrock 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 development. It is expected that the proposed development will be founded on conventional spread footings placed on an undisturbed, hard to very stiff silty clay or a clean, surface sounded bedrock bearing surface.

Bedrock removal may be required for the proposed building excavations depending on the finalized proposed basement and/or underground levels. Bedrock removal may also be required for installation of site services, dependent on the depths of the proposed utilities.

Due to the presence of a silty clay deposit, a permissible grade raise restriction is required for the subject site for all footings founded on a silty clay bearing surface.

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.

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.

Bedrock Removal

As noted above, bedrock removal can be accomplished by hoe ramming where only a small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be completed prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be excavated with almost vertical side walls. A minimum 1 m horizontal ledge should remain between the overburden/weathered bedrock excavation and the sound bedrock surface. The ledge will provide an area to allow for potential sloughing or a stable base for the overburden/weathered bedrock shoring system.

Vibration Considerations

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

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

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to be completed to minimize the risks of claims during or following the construction of the proposed building.

Horizontal Rock Anchors

Bedrock stabilization may be required where the proposed foundation extends into the sound bedrock.

Rock anchors and rock face protection may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

The requirement for rock face protection and rock anchors within the sound bedrock should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

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. If site excavated silty clay, free of organics and deleterious materials, is to be used to build up the subgrade level below paved areas, the silty clay, under dry conditions and above freezing temperatures, should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD using a sheepsfoot roller.

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 Terraxx, connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Value

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on an undisturbed, hard to very stiff brown silty clay can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kP**a and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa** incorporating a geotechnical factor of 0.5.

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.

Footings founded on a clean, surface sounded bedrock can be designed using a bearing resistance value at ULS of **1000 kPa** incorporating a geotechnical factor of 0.5.

A clean surface sounded bedrock 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 a clean, surface sounded bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to silty clay and engineered fill bearing media 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.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

Permissible Grade Raise

A permissible grade raise restriction of **2 m** is recommended for footings placed on a silty clay subgrade within 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

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 (OBC 2012). The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 2 and 3 in Appendix 2 of the present report.

Field Program

The seismic array testing location was placed as presented in Drawing PG5925-1 - Test Hole Location Plan, attached to the present report. Paterson field personnel placed 18 horizontal 2.4 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 1 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 1, 1.5 and 15 m away from the first geophone, 1, 1,5 and 12 m away for the last geophone, and at the centre of the seismic array.

Data Processing and Interpretation

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

The interpretation is repeated at each shot location to provide an average shear wave velocity, 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 the test results, sound bedrock was found to be approximately 4 m below existing grade. The average overburden seismic shear wave velocity was found to be **148 m/s** and the bedrock shear wave velocity was **2,452 m/s**. The V_{s30} was calculated using the standard equation for average shear wave velocity from the Ontario Building as presented below.

Site Class for Footings Founded directly on Bedrock surface

For conventional spread footings bearing directly on bedrock, the V_{s30} was calculated as presented below:

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

$$V_{s30} = \frac{30\ m}{\left(\frac{1\ m}{148\ m/s} + \frac{29\ m}{2{,}452\ m/s}\right)}$$

$$V_{s30} = 1{,}614\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} is **1,614 m/s** for conventional footings founding directly on the bedrock surface. Therefore, a **Site Class A** is applicable for design of proposed buildings in this case, as per Table 4.1.8.4.A of the OBC 2012. Soils underlying the subject site are not susceptible to liquefaction.

Site Class for Footings within 3m of Bedrock Surface

For conventional footings within 3m of bedrock surface, the V_{s30} was calculated as presented below:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{3\ m}{148\ m/s} + \frac{27\ m}{2{,}452\ m/s}\right)}$$

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

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} is **959 m/s** for conventional footings founding within 3m of the bedrock surface. Therefore, a **Site Class B** is applicable for design of proposed buildings in this case, as per Table 4.1.8.4.A of the OBC 2012. Soils underlying the subject site are not susceptible to liquefaction.

Site Class for Footings Greater than 3m Above Bedrock Surface

For conventional footings with more than 3m of softer material between the rock and underside of footing, the V_{s30} was calculated as presented below:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{4\ m}{148\ m/s} + \frac{26\ m}{2{,}452\ m/s}\right)}$$

$$V_{s30=}\ 797\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} is **797 m/s** for conventional footings founding more than 3m of the bedrock surface. Therefore, a **Site Class C** is applicable for design of proposed buildings in this case, as per Table 4.1.8.4.A of the OBC 2012. Soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab / Slab-on-Grade Construction

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

All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD. The upper 300 of the sub-slab fill is recommended to consist of OPSS Granular A crushed stone. If Storage areas are proposed within the underground level, the upper 200 mm of the sub-slab fill should consist of 19 mm clear crushed stone.

Any soft or poor performing areas within the subgrade should be removed and replaced with appropriate backfill material such as OPSS Granular A or Granular B Type II placed in maximum 300 mm thick loose lifts and compacted to 98% of the material's SPMDD. All backfill and compaction efforts should be completed under dry conditions and above freezing temperatures. If winter conditions are expected, refer to Subsection 6.6 for winter construction recommendations.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

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

Lateral Earth Pressure

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 basement 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 exercisedduring the compaction of the backfill materials to stay at least 0.3 m away 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 $0.375 \cdot a \ \cdot \gamma \cdot H^2/g$ where:

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

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

The earthforce component (P_o) under seismic conditions can be calculated using:

> $P = 0.5\ K \cdot \gamma \cdot H^2$, where K = 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 thewall, 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 shouldbe factored as live loads, as per OBC 2012.

If the basement walls are to be poured against a waterproofing system, which will be placed against the exposed bedrock face. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3).

Where soil is retained, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees anda drained unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total staticearth pressure when using the effective unit weight.

5.7 Pavement Structure

Car only parking areas and heavy traffic access areas are expected at this site. The subgrade material will consist of native soil, fill and possibly bedrock. The proposed pavement structures are presented in Tables 5 and 6.

Table 5 – Recommended Pavement Structure – Asphalt Surfaced 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, bedrock or concrete fill.	

Table 4 – Recommended Pavement Structure – Asphalt Surfaced Access Lanes and Heavy Truck Parking/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 fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, bedrock or concrete fill.	

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.

Pavement Structure Drainage

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

Due to the impervious nature of the silty clay deposit, where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction. The subdrain inverts should be approximately 300 mm below subgrade level and run longitudinal along the curb lines. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. The system should consist of a 100 to 150 mm diameter perforated, corrugated plastic pipe which is surrounded on all sides by 150 mm of 19 mm clear crushed stone and is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Waterproofing of the foundation walls may be required if more than one underground level is anticipated. Due to the lack of bedrock coring, the groundwater table depth was not accurately measure below the bedrock surface. However, based on the current information, waterproofing is not anticipated to be required if one underground level is being considered. The requirement for waterproofing should be confirmed by Paterson upon commencement of excavation when the groundwater infiltration can be better assessed.

Underfloor Drainage

Underfloor drainage is recommended to control water infiltration due to groundwater infiltration at the proposed founding elevation. For preliminary design purposes, Paterson recommends that 150 mm in diameter perforated pipes be placed at approximately 6 m centers. 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 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, as well as structures that will be subjected to exterior conditions for an extended period of time (such as the underground parking entrance), 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.

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. It is assumed that sufficient room will be available for the greater part of the excavation to 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 Excavations

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

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

It is recommended that a trench box 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 following parameters.

Table 7 – 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 99% 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 clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

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.

Clay Seals

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. The seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material.

The seals should consist of relatively dry and compactable 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

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.

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 an aggressive to very aggressive corrosive environment.

6.8 Landscaping Considerations

The proposed development is located in an area of low to medium sensitive silty clay deposits for tree planting. Based on our review of the subsurface profile below the subject site, the underlying silty clay deposit is relatively dry and designated as a very stiff to firm silty clay. Therefore, the proposed development is considered to be located within an area of low sensitive silty clay deposits for tree planting.

Tree Planting Restrictions

Based on the results of the representative soil samples, the subject site is considered as a low/medium sensitivity area for tree planting according to the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines)

Since the modified plasticity limit (PI) generally does not exceed 40%, large trees (mature height over 14 m) can be planted at the subject site provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space).

Based on our testing results, tree planting setback limits should be 4.5 m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m) provided that the following conditions are met:

- ❑ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the centre of the tree trunk and verified by means of the Grading Plan as indicated procedural changes below.

- ❑ A small tree must be provided with a minimum of 25 m3 of available soil volume while a medium tree must be provided with a minimum of 30 m3 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 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.

- ❑ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).

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

It is important to note that is the building is founded on bedrock, the above noted tree planting restriction will not apply. However, exterior structures such as canopy footings or any settlement sensitive structures founded on silty clay will be subjected to the above noted tree planting setbacks.

7.0 Grading Plan Review

The following grading plan drawing, prepared by LRL Engineering, has been reviewed by Paterson in preparation for the current memorandum:

- Grading and Drainage Plan – Orleans Residential and Medical Facility 3996 Innes Road, Ottawa, ON – Project No. 230737 – Drawing No. C301 – Revision 4 dated November 6, 2024.

Permissible grade Raise

Based on our review of the above noted grading plan, the proposed grade raises within the aforementioned site are within the recommended permissible grade raise of 2.0 m. No exceedances were noted for any area within the subject site. Therefore, the proposed grade raises are generally acceptable from a geotechnical perspective and will not require the use of lightweight fill at this time.

Protection of Footings Against Frost Action

Based on our review, all of the proposed footings will have sufficient soil cover except the footings at the underground parking entrance location and the footings of the proposed retaining walls along the entrance ramp.

It is recommended that HI-40 rigid insulation with a minimum thickness of 150 mm, or approved equivalent, be placed directly below the underside of the ramp wall footings. It is further recommended that SM rigid insulation with a minimum thickness of 50 mm be placed horizontally below the entire area of the ramp slabs.

If a heated ramp is proposed using glycol lines, it is recommended that an alarm system be installed to notify the maintenance team if the system is not operational.

The SM rigid insulation should be extended horizontally a minimum of 600 mm beyond the exterior face of the footings.

For building footings located at the entrance of the buildings' garages, it is recommended that 150 mm of HI-40 rigid insulation, or approved equivalent, be placed below the buildings' footings A minimum 1500 mm SM rigid insulation should extend horizontally beyond the footing face into the direction of the ramp and a minimum 900 mm to extend horizontally into the garage direction.

Reference Should be made to Figure 4 - Cross Section at the Entrance of Garage for Building and Figure 5 - Frost Protection Recommendation for Retaining Walls and Ramps for the details of frost protection and Figure 6 - Marked-up Plans for The Location and Placement of Rigid Insulation, attached to the current memorandum.

Table 8 – Frost Protection Recommendations for Footings with Reduced Soil Cover

Thermal Condition	Soil Cover Provided (mm)	Insulation Dimensions	
		Thickness (mm)	Extension (mm)
Unheated	1800-2100	50	Extend 900 mm horizontally beyond the exterior edge of the footing face and 600 mm beyond the interior edge of the footing face.
	1200-1800	50	Extend 600 mm horizontally beyond the exterior edge of the footing face and 600 mm beyond the interior edge of the footing face.
	900-1200	75	Extend 1200 mm horizontally beyond the exterior edge of the footing face and 600 mm beyond the interior edge of the footing face.
	600-900	100	Extend 1800 mm horizontally beyond the exterior edge of the footing face and 600 mm beyond the interior edge of the footing face.
	300-600	150	Extend 2100 mm horizontally beyond the exterior edge of the footing face and 600 mm beyond the interior edge of the footing face.
	0-300	200	Extend 2100 mm horizontally beyond the exterior edge of the footing face and 600 mm beyond the interior edge of the footing face.

Note:
The rigid insulation thicknesses and extensions provided herein are site specific and should not be used on other sites without consulting Paterson Group for the sufficiency of the provided recommendations.

Rigid insulation should consist of HL-40 or equivalent and the rigid insulation boards should be placed below the proposed footings upon a level and flat surface and with no gaps between abutting boards. Consideration can be given to placing a thin leveling mat consisting of a layer of compacted OPSS Granular A crushed stone, stone dust, or sand below the insulation layer, as required. SM Rigid insulation can be used beyond the footing face in the same manner provided for the HI40 rigid insulation.

Parking Garage Ramp Drainage System

It is recommended that the following drainage system be implemented below the ramps:

- A minimum 150 mm diameter perforated, corrugated drainage pipe be placed within the centre of the ramp or adjacent to the ramp walls and the garage entrance footings along the founding elevation. The drainage pipe is also recommended to be wrapped with a filter cloth and a 300 mm layer of clear crushed stone.
- The drainage pipe should have positive drainage toward an outlet and be mechanically connected to the building's underfloor drainage system. The subgrade below the ramp should be shaped to allow for positive drainage of any accumulated water towards the drainage pipes (minimum 1% slope).
- The area between the subgrade level and the underside of the concrete ramp should be backfilled with free draining, non-frost susceptible granular fill such as OPSS Granular A or Granular B Type II compacted to 98% of the material's SPMDD. The granular fill should be placed in max 300 mm thick loose lifts and compacted to a minimum 98% of the material's SPMDD.
- The above-noted work should be reviewed and approved by Paterson at the time of construction.

It should be noted that the USF elevation of the proposed retaining walls at west and east side of the site is not shown on the above noted grading plan drawing. Therefore, if insufficient soil cover is provided for footings, rigid insulation should be installed for the proposed retaining walls as recommended in the table below.

Subgrade Preparations

The rigid insulation to be placed below the proposed footings and ramp structures should be placed on a level bearing surface reviewed and approved by Paterson personnel at the time of construction. Consideration should be taken for placing a thin levelling mat consisting of a layer of compacted OPSS Granular A crushed stone, stone dust or clear crushed stone material. The thickness of the levelling mat should not exceed a thickness of 50 mm.

Winter Construction

If the construction of the subject retaining walls and ramps is anticipated to take place during the winter months, it is further recommended that the remaining portion of the excavation, where settlement sensitive structures will be constructed at a later date, be covered with 50 mm of SM rigid insulation until the area is exposed for construction. Reference should be made to Subsection 6.6 of the current report for complete winter construction recommendations.

Field Inspections

The above noted recommendations should be reviewed and approved in the field by Paterson personnel. Paterson should be provided with the finalized design drawings once available to prepare for field inspections.

8.0 Recommendations

It is recommended that the following be carried out by Paterson once final detailed design of the proposed development has been prepared:

- ❑ Updating the grading and servicing review upon changes to the grading and servicing plans.
- ❑ Review of the foundation drainage and waterproofing design, if not designed by Paterson.
- ❑ Review of the excavation plan and temporary shoring design, if not designed by Paterson.

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:

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

A report confirming 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*****.**

9.0 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 Mr. Loutfi Frangian 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.

Owen R. Canton, B.Eng.

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ❑ Mr. Loutfi Frangian (e-mail copy)
- ❑ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE DISTRIBUTION AND HYDROMETER TESTING RESULTS

ATTERBERG LIMIT TESTING RESULTS

SHRINKAGE TESTING RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 3996 Innes Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE August 18, 2021

FILE NO. PG5925

HOLE NO. BH 1-21

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 % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	89.79		
TOPSOIL 0.08									
FILL: Brown silty sand, trace topsoil 0.56		AU	1						
Hard to very stiff, brown **SILTY CLAY**		SS	2	75	13	1	88.79		
		SS	3	75	13	2	87.79		
2.29									
End of Borehole Practical refusal to augering at 2.29m depth. (BH dry upon completion)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 3996 Innes Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE August 18, 2021

FILE NO. PG5925

HOLE NO. BH 1B-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.76
OVERBURDEN						1	88.76
						2	87.76
2.29							
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders, trace clay		SS	1	20	50+		
2.84							
End of Borehole Practical refusal to augering at 2.84m depth. (GWL @ 2.83m - August 20, 2021)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - 3996 Innes Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE August 18, 2021

FILE NO. PG5925

HOLE NO. BH 2-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	90.35
TOPSOIL 0.08							
FILL: Brown silty sand 0.60		AU	1				
Hard to very stiff, brown **SILTY CLAY**		SS	2	75	11	1	89.35
		SS	3	83	11	2	88.35
2.39		SS	4	50	50+		
End of Borehole							

Practical refusal to augering at 2.39m depth.

(GWL @ ground surface - August 20, 2021)

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

○ Water Content %

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

Piezometer Construction

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

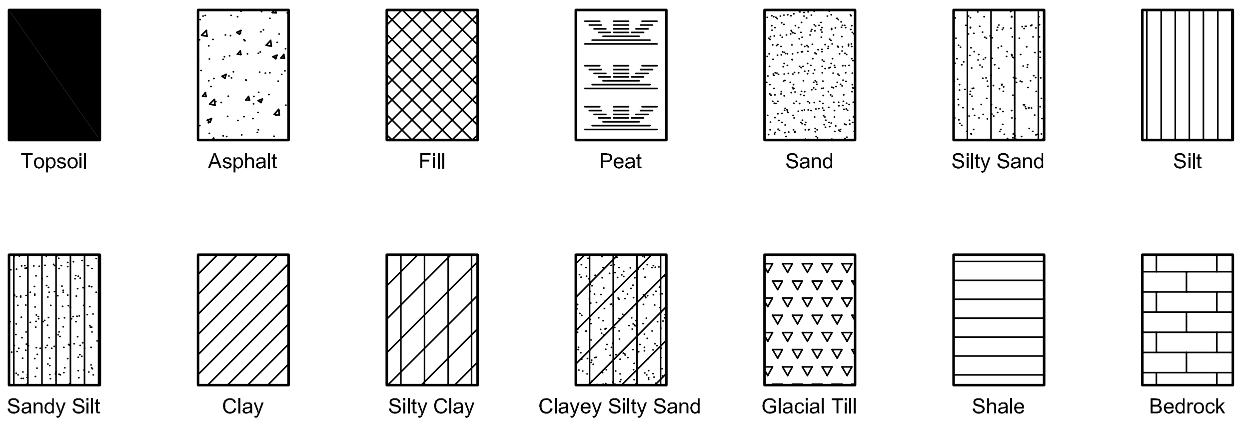
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

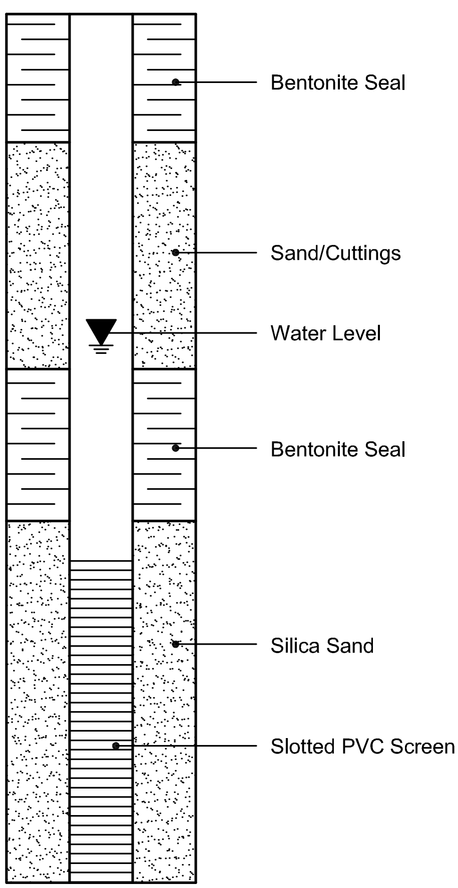
SYMBOLS AND TERMS (continued)

STRATA PLOT

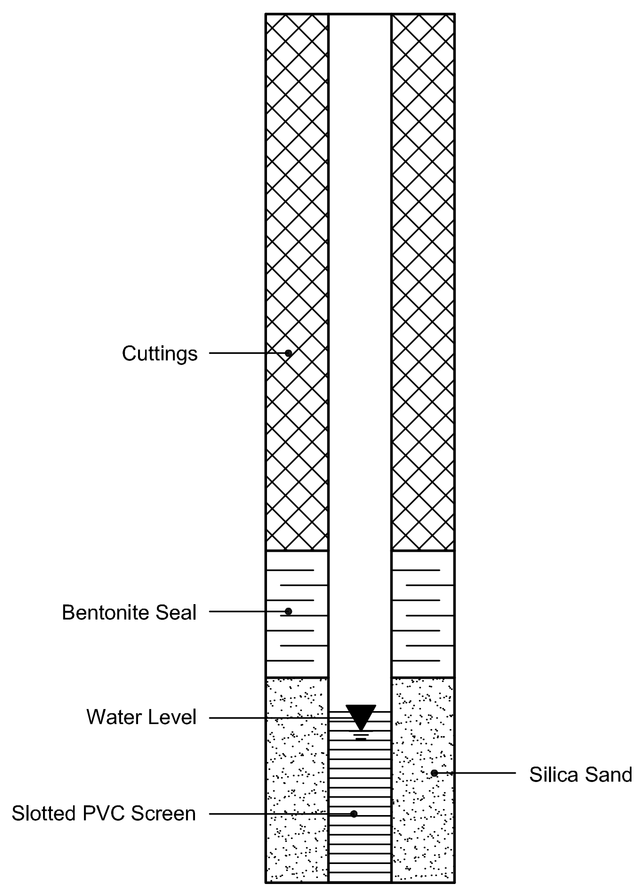


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



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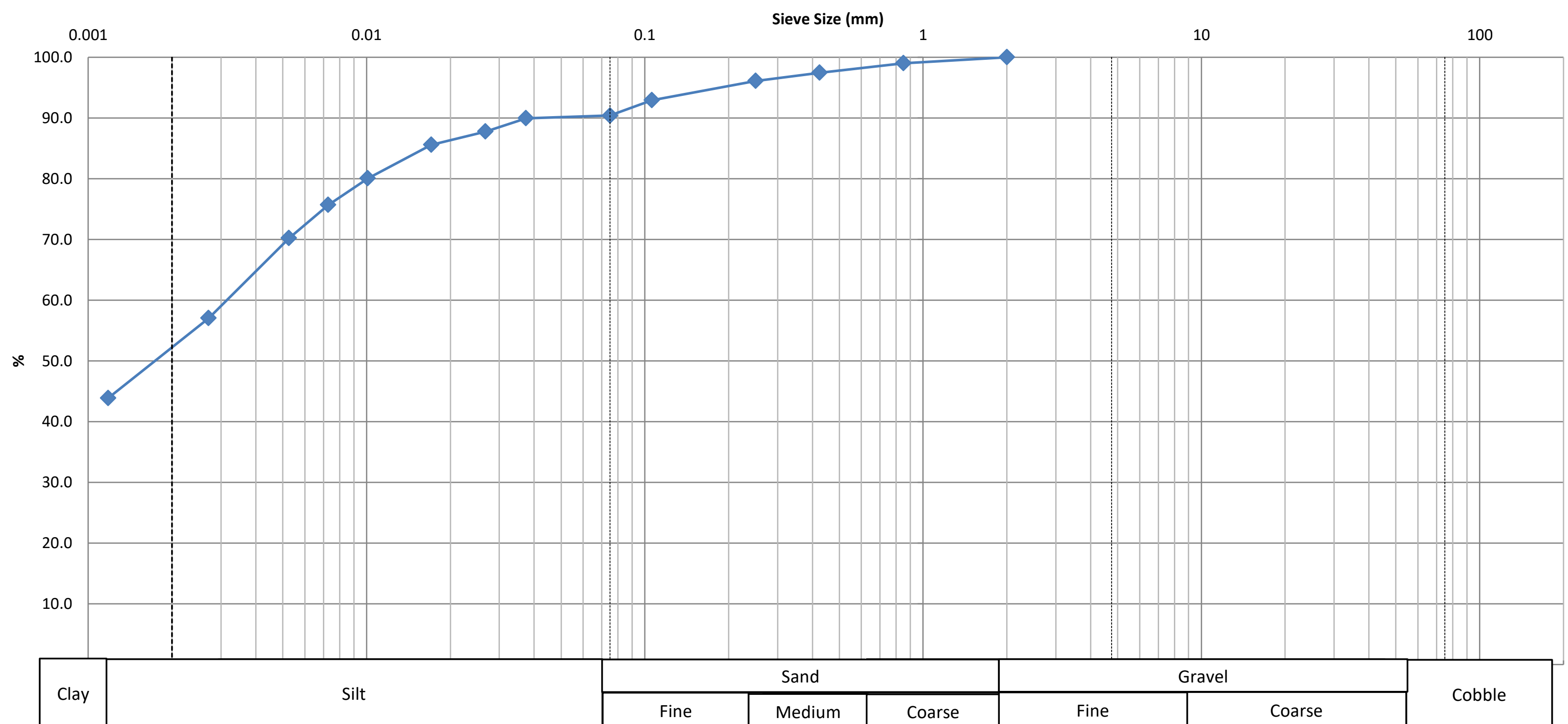
SIEVE ANALYSIS
ASTM C136

CLIENT:	Mr Loufti Frangian	DEPTH:	2' 6" - 4' 6"	FILE NO:	PG5925
CONTRACT NO.:		BH OR TP No.:	BH2-21 SS2	LAB NO:	28298
PROJECT:	3996 Innes Road			DATE RECEIVED:	2-Sep-21
				DATE TESTED:	8-Sep-21
DATE SAMPLED:	2-Sep-21			DATE REPORTED:	10-Sep-21
SAMPLED BY:	J.K			TESTED BY:	DB



Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						30.4					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					0.0	9.6		38.4		52.0	

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

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HYDROMETER
LS-702 ASTM-422

CLIENT:	Mr Loufti Frangian	DEPTH:	2' 6" - 4' 6"	FILE NO.:	PG5925
PROJECT:	3996 Innes Road	BH OR TP No.:	BH2-21 SS2	DATE SAMPLED	2-Sep-21
LAB No. :	28298	TESTED BY:	DB	DATE RECEIVE	2-Sep-21
SAMPLED BY:	J.K	DATE REPT'D:	10-Sep-21	DATE TESTED:	8-Sep-21

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
117.6	2.700

INITIAL WEIGHT	50.00	**HYGROSCOPIC MOISTURE**		
WEIGHT CORRECTED	44.47	TARE WEIGHT	50.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	4.83	AIR DRY	150.00	100.00
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	138.94	88.94
		CORRECTED	0.889	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75			
2.0	**0.0**	0.0	**100.0**
Pan	**117.6**		
0.850	**0.49**	1.0	**99.0**
0.425	**1.27**	2.5	**97.5**
0.250	**1.96**	3.9	**96.1**
0.106	**3.54**	7.1	**92.9**
0.075	**4.80**	9.6	**90.4**
Pan	**4.83**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	9:32	47.0	6.0	23.0	0.0374	89.9	**89.9**
2	9:33	46.0	6.0	23.0	0.0267	87.8	**87.8**
5	9:36	45.0	6.0	23.0	0.0171	85.6	**85.6**
15	9:46	42.5	6.0	23.0	0.0101	80.1	**80.1**
30	10:01	40.5	6.0	23.0	0.0073	75.7	**75.7**
60	10:31	38.0	6.0	23.0	0.0053	70.2	**70.2**
250	1:41	32.0	6.0	23.0	0.0027	57.0	**57.0**
1440	9:31	26.0	6.0	23.0	0.0012	43.9	**43.9**

COMMENTS:

Moisture = 30.36%

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.

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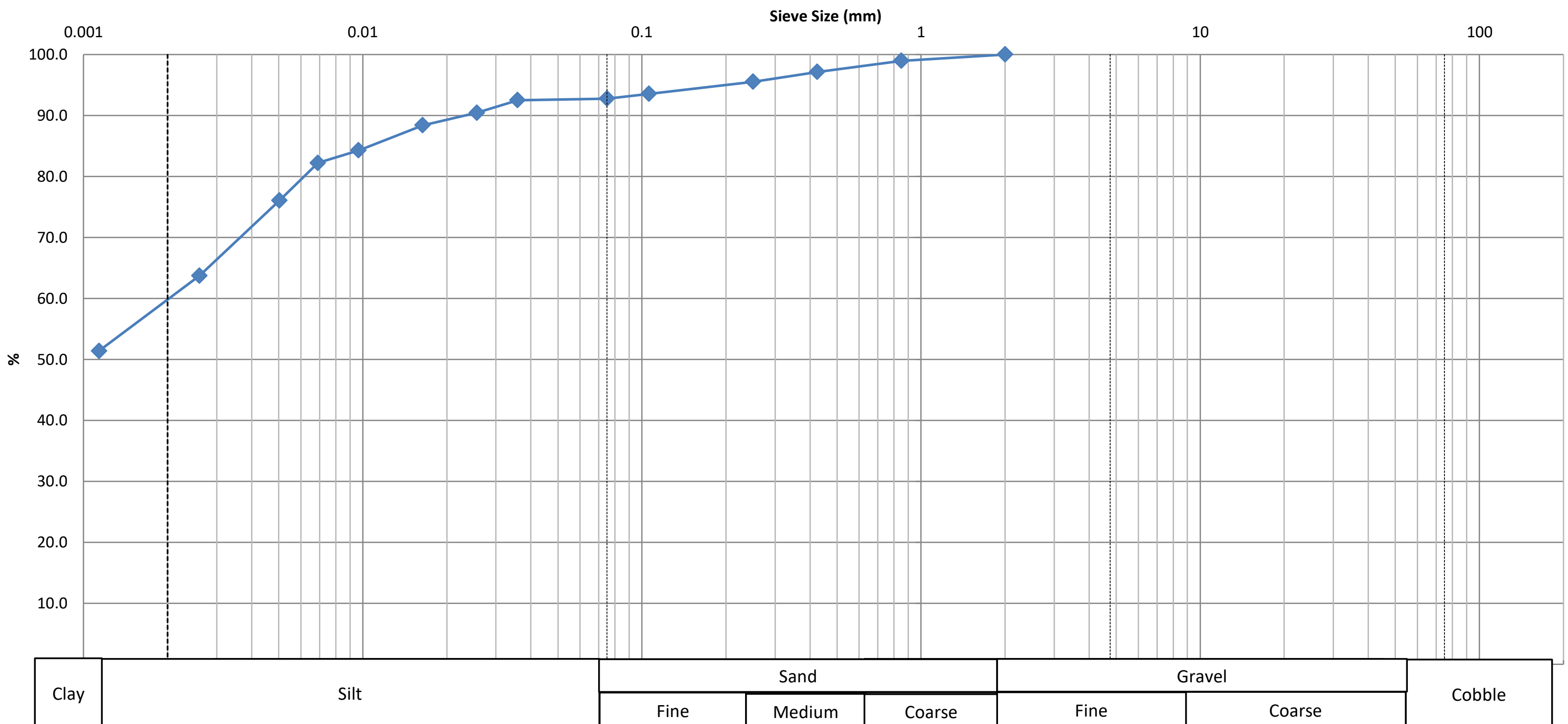
SIEVE ANALYSIS
ASTM C136

CLIENT:	Mr. Loufti Frangian	DEPTH:	2' 4" - 4' 6"	FILE NO:	PG5925
CONTRACT NO.:		BH OR TP No.:	BH18-21 SS2	LAB NO:	28297
PROJECT:	3996 Innes Road			DATE RECEIVED:	2-Sep-21
				DATE TESTED:	8-Sep-21
DATE SAMPLED:	2-Sep-21			DATE REPORTED:	10-Sep-21
SAMPLED BY:	JK			TESTED BY:	DB



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					35.1					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					0.0	7.2	32.8	60.0		

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

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HYDROMETER
LS-702 ASTM-422

CLIENT:	Mr. Loufti Frangian	DEPTH:	2' 4" - 4' 6"	FILE NO.:	PG5925
PROJECT:	3996 Innes Road	BH OR TP No.:	BH18-21 SS2	DATE SAMPLED	2-Sep-21
LAB No. :	28297	TESTED BY:	DB	DATE RECEIVE	2-Sep-21
SAMPLED BY:	JK	DATE REPT'D:	10-Sep-21	DATE TESTED:	8-Sep-21

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
114.1	2.700

INITIAL WEIGHT	50.00	**HYGROSCOPIC MOISTURE**		
WEIGHT CORRECTED	43.37	TARE WEIGHT	50.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	3.67	AIR DRY	150.00	100.00
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	136.74	86.74
		CORRECTED	0.867	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19			
13.2			
9.5			
4.75			
2.0	**0.0**	0.0	**100.0**
Pan	**137.6**		
0.850	**0.51**	1.0	**99.0**
0.425	**1.43**	2.9	**97.1**
0.250	**2.23**	4.5	**95.5**
0.106	**3.23**	6.5	**93.5**
0.075	**3.62**	7.2	**92.8**
Pan	**3.67**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	9:20	51.0	6.0	23.0	0.0359	92.5	**92.5**
2	9:21	50.0	6.0	23.0	0.0256	90.4	**90.4**
5	9:24	49.0	6.0	23.0	0.0164	88.4	**88.4**
15	9:34	47.0	6.0	23.0	0.0097	84.3	**84.3**
30	9:49	46.0	6.0	23.0	0.0069	82.2	**82.2**
60	10:19	43.0	6.0	23.0	0.0050	76.1	**76.1**
250	1:29	37.0	6.0	23.0	0.0026	63.7	**63.7**
1440	9:19	31.0	6.0	23.0	0.0011	51.4	**51.4**

COMMENTS:

Moisture = 35.12%

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.

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ATTERBERG LIMITS
LS-703/704

CLIENT:	Mr. Loutfi Frangian	FILE NO.:	PG5925
PROJECT:	3996 Innes Road	DATE SAMPLED:	19-Aug-21
LOCATION:	BH1-21 SS3 @ 5' - 7'	DATE REPORTED:	26-Aug-21

LIQUID LIMIT DETERMINATION

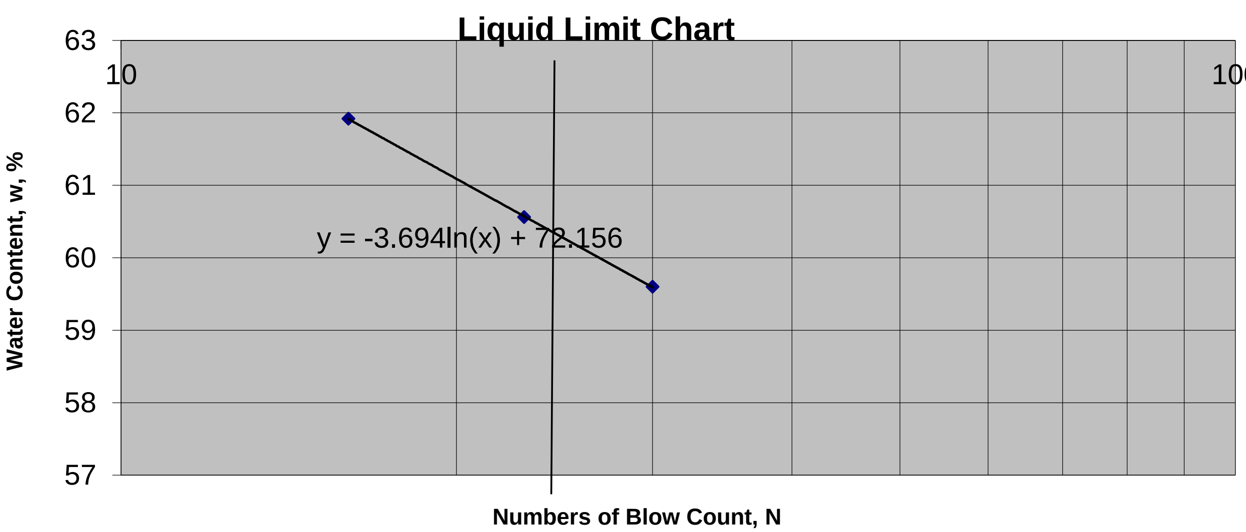
CAN NO.	**2**	**3**	**4**				
WT. OF CAN	8.65	8.64	8.66				
WT. OF SOIL & CAN	21.83	18.98	18.97				
WT. OF DRY SOIL & CAN	16.79	15.08	15.12				
WT. OF MOISTURE	5.04	3.9	3.85				
WT. OF DRY SOIL & CAN	8.14	6.44	6.46				
WATER CONTENT, w, %	**61.92**	**60.56**	**59.6**				
NO. OF BLOWS, N	16	23	30				

PLASTIC LIMIT DETERMINATION

CAN NO.	**10**	**11**
WT. OF CAN	19.81	20.00
WT. OF SOIL & CAN	26.98	26.86
WT. OF DRY SOIL & CAN	25.44	25.42
WT. OF MOISTURE	1.54	1.44
WT. OF DRY SOIL & CAN	5.63	5.42
WATER CONTENT, w, %	**27.35**	**26.57**

RESULTS

LIQUID LIMIT	**61**
PLASTIC LIMIT	**27**
PLASTICITY INDEX	**34**



TECHNICIAN: DB	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

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ATTERBERG LIMITS
LS-703/704

CLIENT:	Mr. Loutfi Frangian	FILE NO.:	PG5925
PROJECT:	3996 Innes Road	DATE SAMPLED:	19-Aug-21
LOCATION:	BH2-21 SS3 @ 5' - 7'	DATE REPORTED:	26-Aug-21

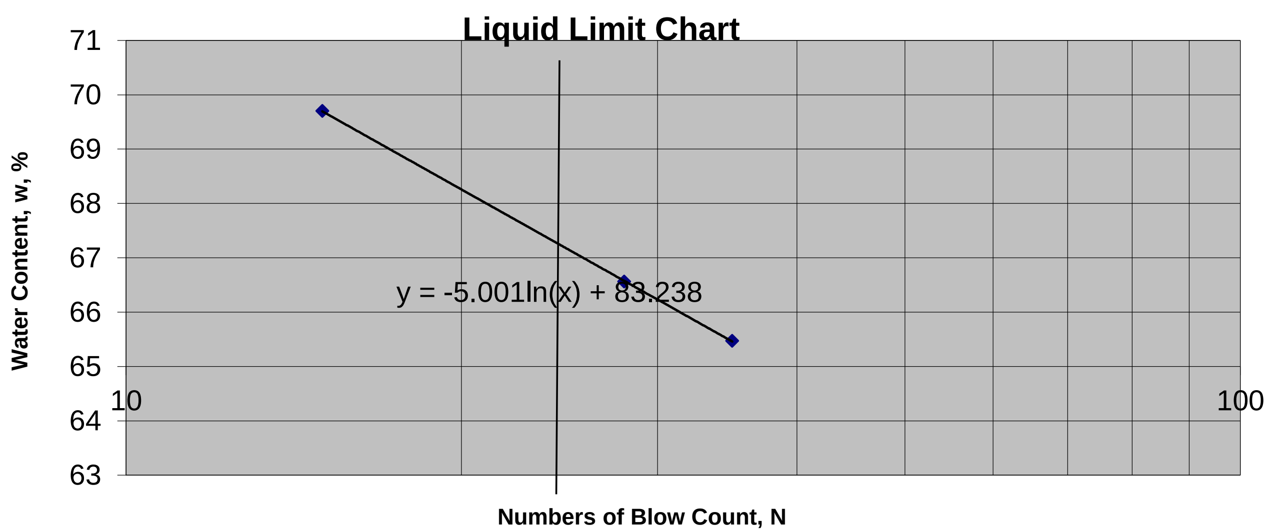
LIQUID LIMIT DETERMINATION

CAN NO.	**33**	**34**	**35**				
WT. OF CAN	4.34	4.34	4.42				
WT. OF SOIL & CAN	13.86	15.00	15.44				
WT. OF DRY SOIL & CAN	9.95	10.74	11.08				
WT. OF MOISTURE	3.91	4.26	4.36				
WT. OF DRY SOIL & CAN	5.61	6.4	6.66				
WATER CONTENT, w, %	**69.7**	**66.56**	**65.47**				
NO. OF BLOWS, N	15	28	35				

PLASTIC LIMIT DETERMINATION

CAN NO.	**1**	**2**
WT. OF CAN	19.87	19.95
WT. OF SOIL & CAN	26.81	27.22
WT. OF DRY SOIL & CAN	25.14	25.50
WT. OF MOISTURE	1.67	1.72
WT. OF DRY SOIL & CAN	5.27	5.55
WATER CONTENT, w, %	**31.69**	**30.99**

LIQUID LIMIT	**67**
PLASTIC LIMIT	**31**
PLASTICITY INDEX	**36**



TECHNICIAN: DB	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

patersongroup
consulting engineers

Linear Shrinkage
ASTM D4943-02

CLIENT:	Dominion Lending Centre	DEPTH	2'6"-4'6"	FILE NO.:	PG 5914
PROJECT:	Borehole Drilling	BH OR TP No:	BH2-21 SS2	DATE SAMPLED	02-Sep
LAB No:	28296	TESTED BY:	DB	DATE RECEIVED	02-Sep
SAMPLED BY:	IK	DATE REPORTED:		DATE TESTED	03-Sep

LABORATORY INFORMATION & TEST RESULTS

Moisture	No. of Blows(4)	Calibration (Two Trials)	Tin NO.(x15)	
Tare	4.84	Tin	4.77	4.77
Soil Pat Wet + Tare	68.6	Tin + Grease	4.84	4.84
Soil Pat Wet	63.76	Glass	48.97	48.97
Soil Pat Dry + Tare	53.01	Tin + Glass + Water	91.20	91.20
Soil Pat Dry	48.17	Volume	37.39	37.39
Moisture	**32.36**	**Average Volume**	**37.39**	

Soil Pat + String	48.18
Soil Pat + Wax + String in Air	54.52
Soil Pat + Wax + String in Water	23.76
Volume Of Pat (Vdx)	30.76

RESULTS:

Shrinkage Limit	**3.81**
Shrinkage Ratio	**2.038**
Volumetric Shrinkage	**58.188**
Linear Shrinkage	**14.175**

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 24531

Report Date: 27-Aug-2021

Order Date: 24-Aug-2021

Project Description: PG5925

	Client ID:	BH2-21/SS3	-	-	-
	Sample Date:	19-Aug-21 09:00	-	-	-
	Sample ID:	2135373-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	73.2	-	-	-
General Inorganics					
pH	0.05 pH Units	6.75	-	-	-
Resistivity	0.10 Ohm.m	18.8	-	-	-
Anions					
Chloride	5 ug/g dry	260	-	-	-
Sulphate	5 ug/g dry	53	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURE 4 – CROSS SECTION AT THE ENTRANCE OF GARAGE FOR BUILDING

FIGURE 5 – FROST PROTECTION RECOMMENDATION FOR RETAINING WALLS AND RAMPS

FIGURE 6 – MARKED-UP PLANS FOR THE LOCATION AND PLACEMENT OF RIGID INSULATION

DRAWING PG5925-1 - TEST HOLE LOCATION PLAN

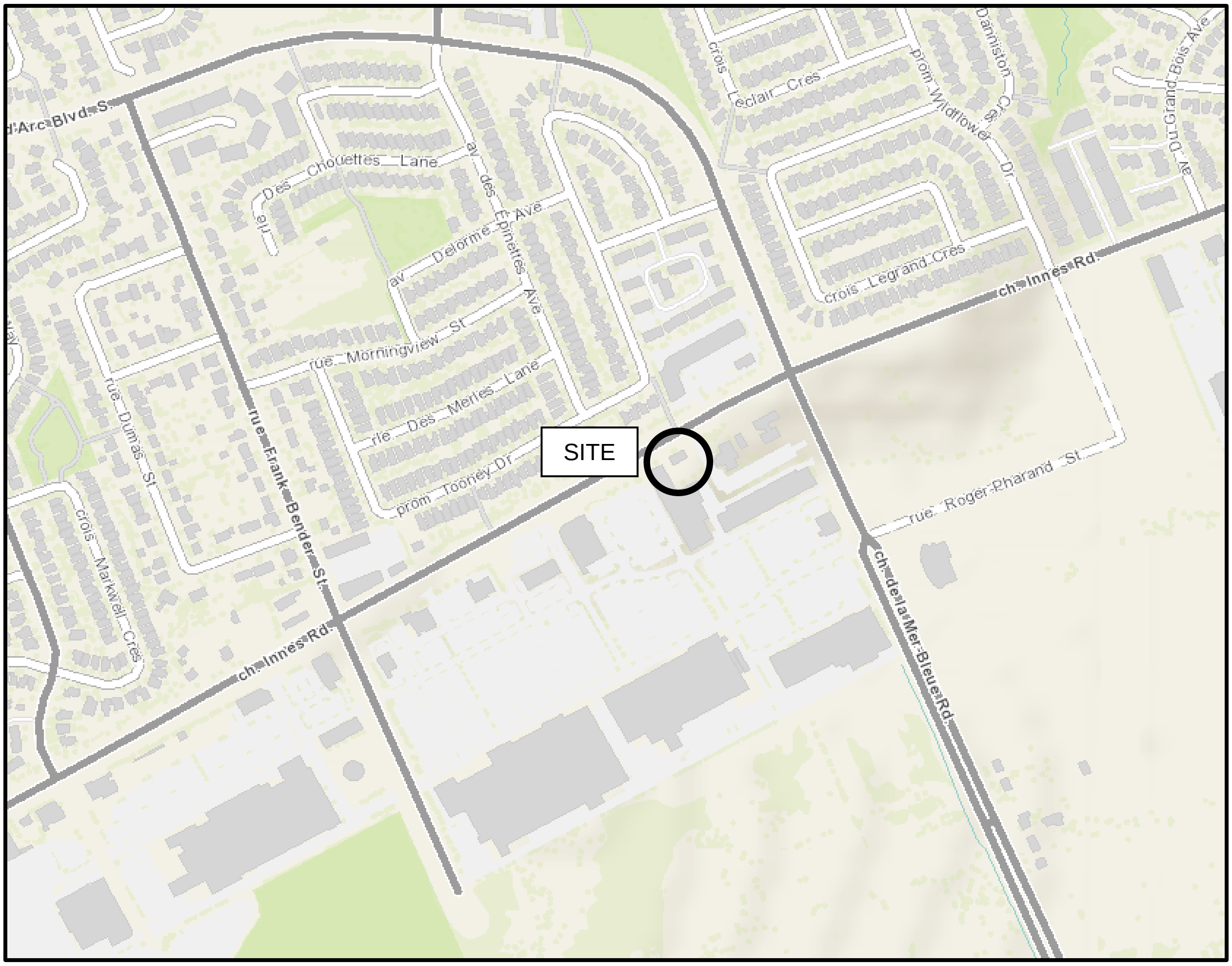


FIGURE 1

KEY PLAN

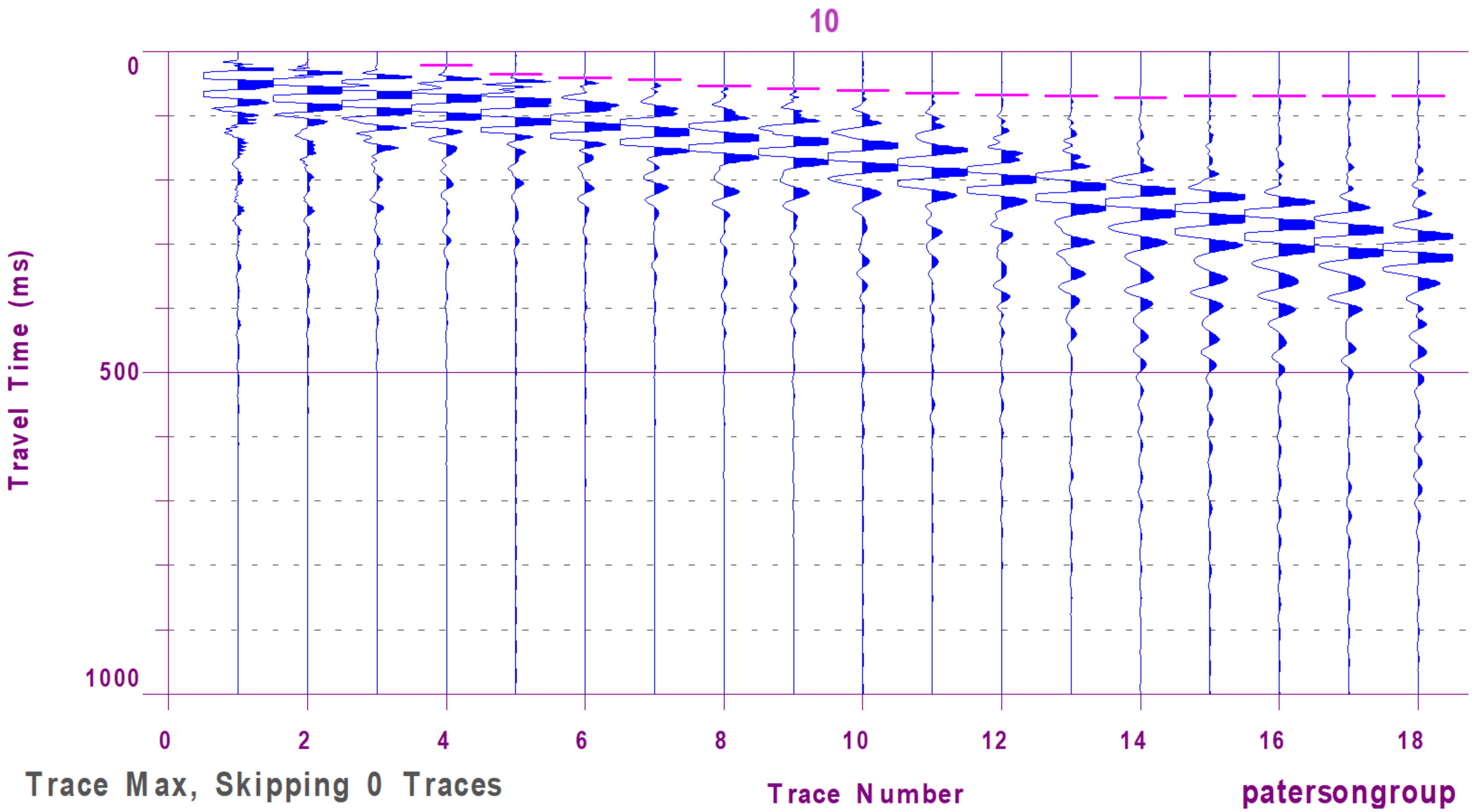


Figure 2 – Shear Wave Velocity Profile at Shot Location -1 m

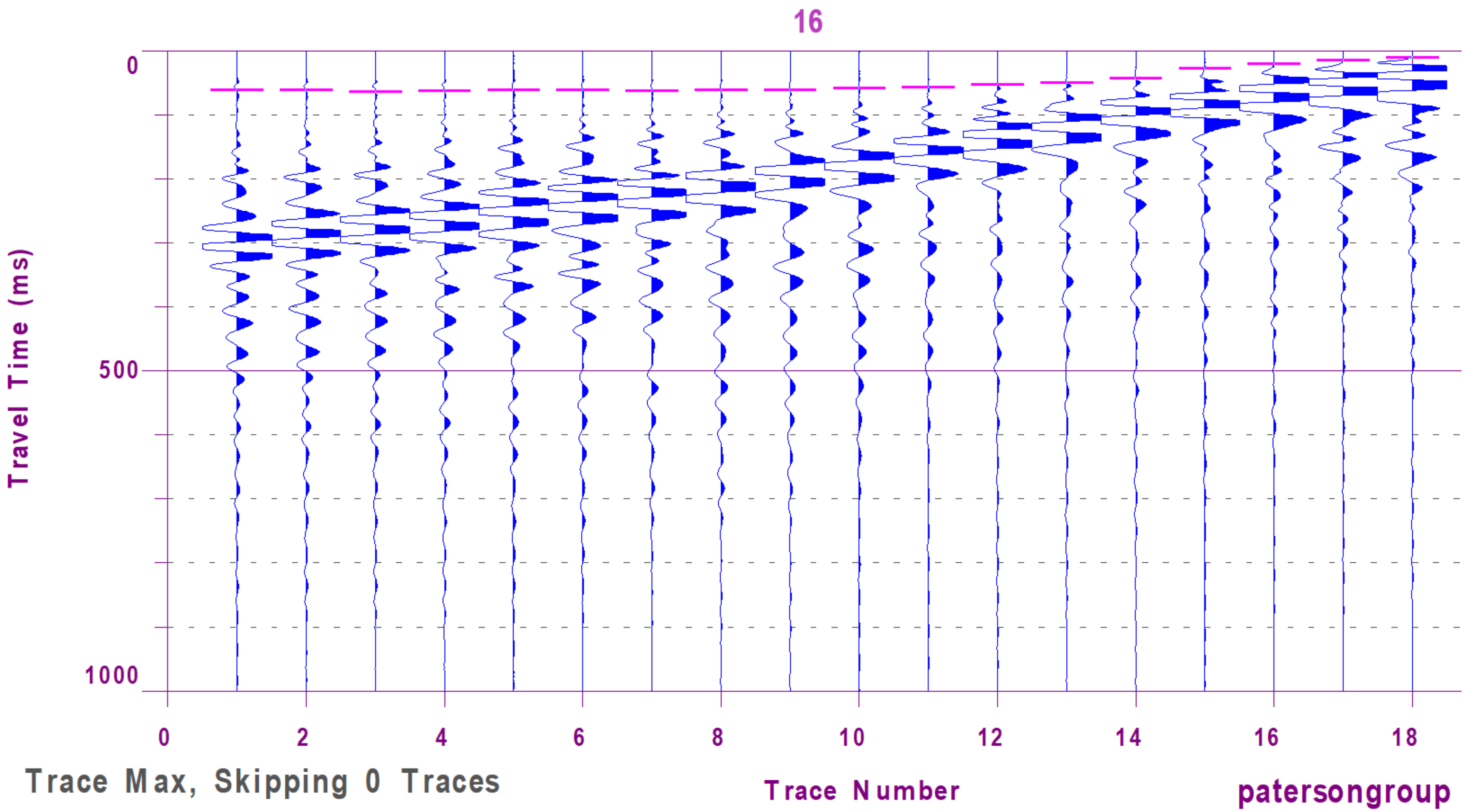
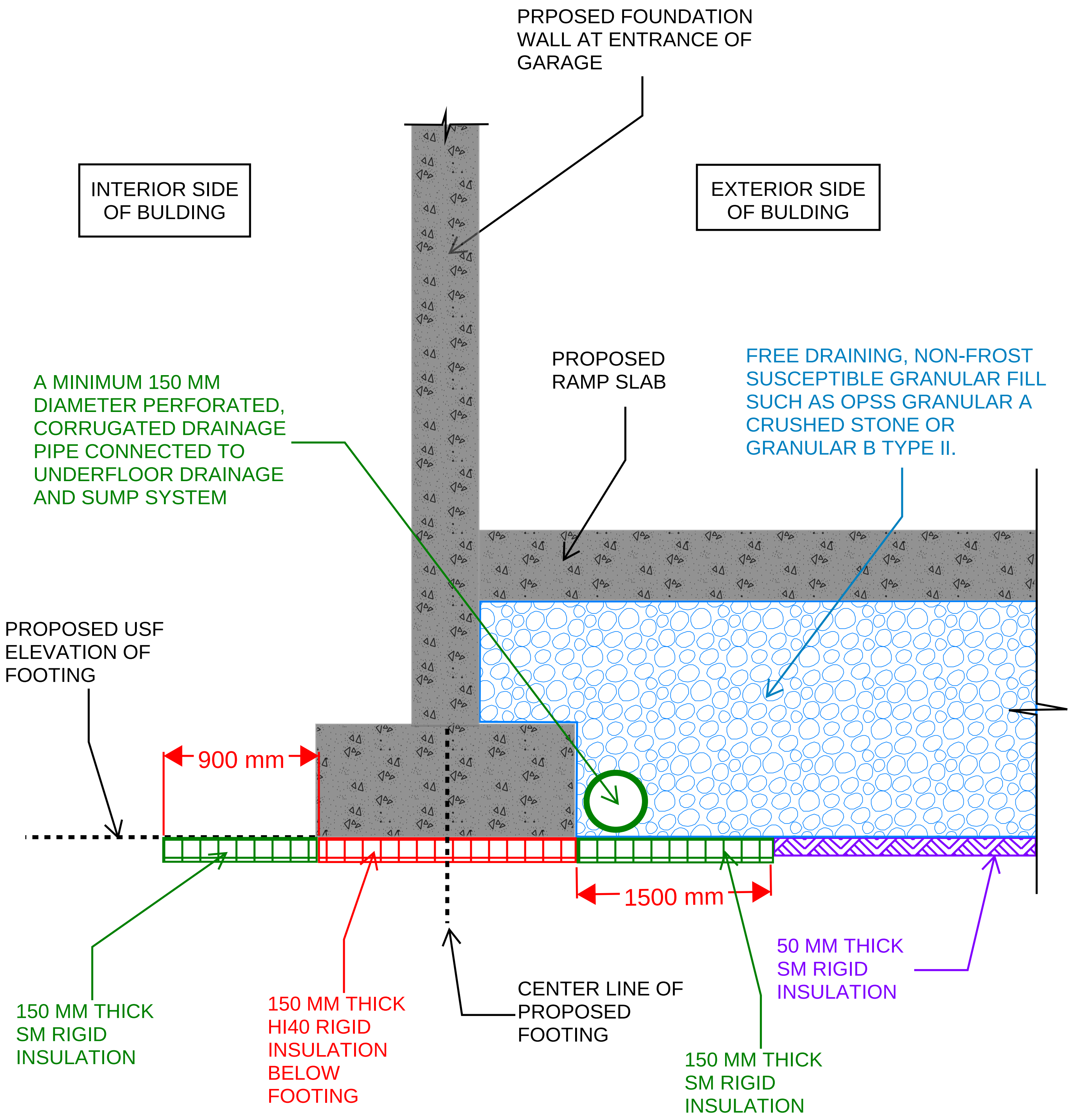


Figure 3 – Shear Wave Velocity Profile at Shot Location 18.5 m

PATERSON GROUP

PATERSON GROUP	Frost Protection Recommendations for Proposed Retaining Wall and Ramp	Job No.: PG5925	
	3996 Innes Road, Ottawa, ON	Date: February 6, 2025	
	Figure 4 - Cross Section at the Entrance of Garage for Building	Drawn by: YZ	Approved by: FA
	Drawing is not to scale	Page: 1 of 1	



PATERSON GROUP	Frost Protection Recommendations for Proposed Retaining Wall and Ramp	Job No.: PG5925	
	3996 Innes Road, Ottawa, ON	Date: February 6, 2025	
	Figure 5 - Frost Protection Recommendation for Retaining Walls and Ramps	Drawn by: YZ	Approved by: FA
	Drawing is not to scale	Page: 1 of 1	

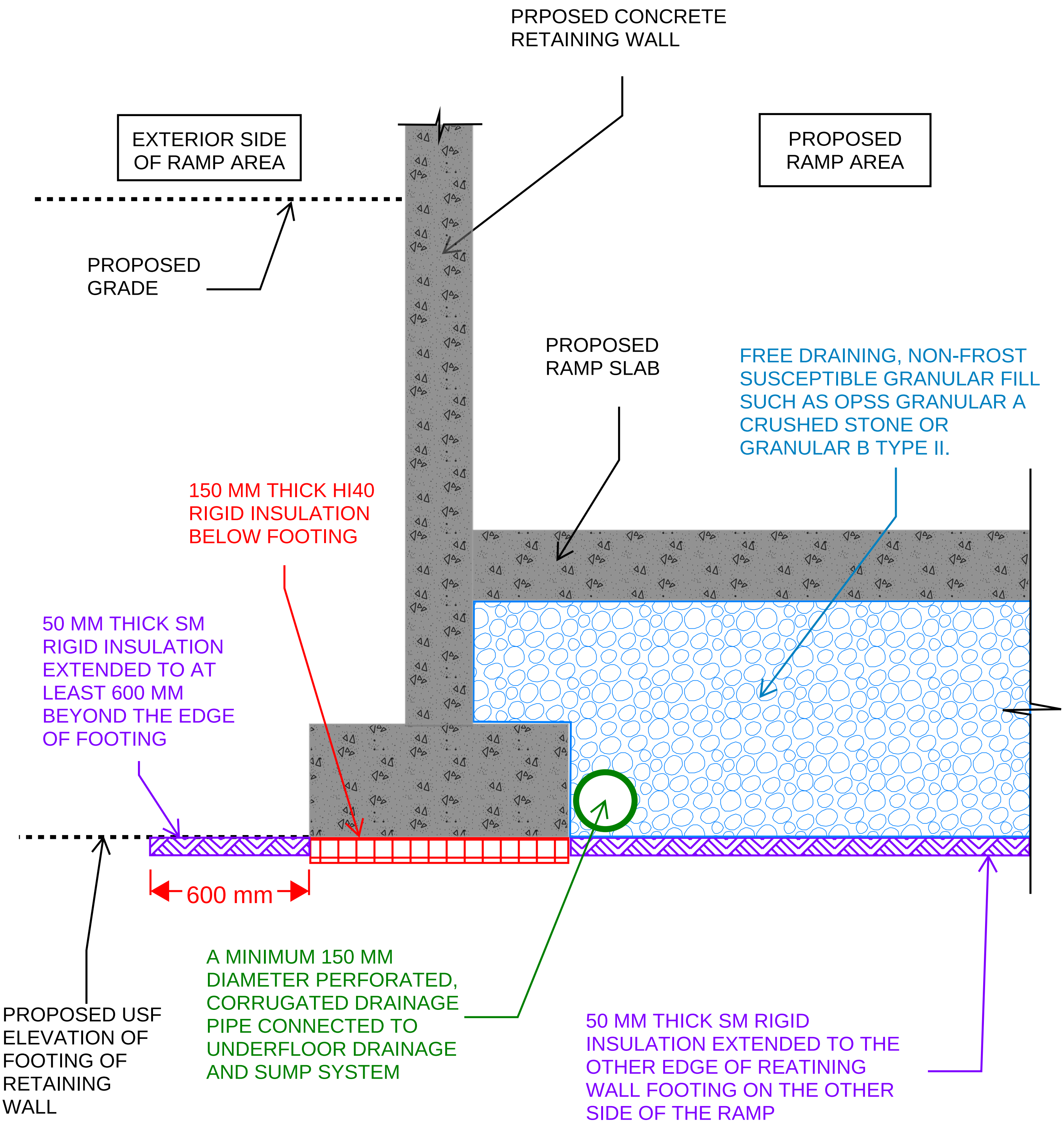
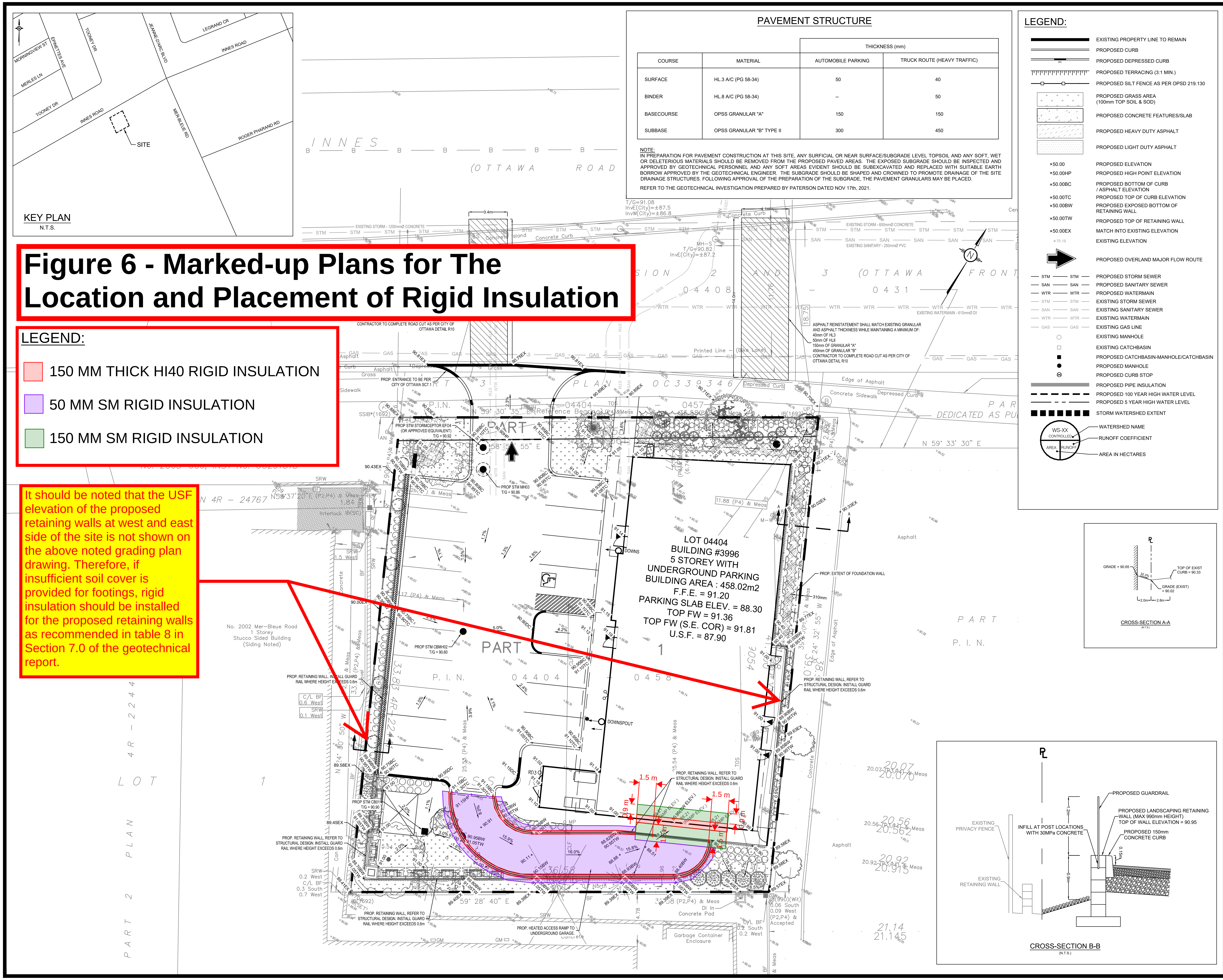


Figure 6 - Marked-up Plans for The Location and Placement of Rigid Insulation



PAVEMENT STRUCTURE

COURSE	MATERIAL	THICKNESS (mm) AUTOMOBILE PARKING	THICKNESS (mm) TRUCK ROUTE (HEAVY TRAFFIC)
SURFACE	HL.3 A/C (PG 58-34)	50	40
BINDER	HL.8 A/C (PG 58-34)	--	50
BASECOURSE	OPSS GRANULAR "A"	150	150
SUBBASE	OPSS GRANULAR "B" TYPE II	300	450

NOTE:
IN PREPARATION FOR PAVEMENT CONSTRUCTION AT THIS SITE, ANY SURFICIAL OR NEAR SURFACE/SUBGRADE LEVEL TOPSOIL AND ANY SOFT, WET OR DELETERIOUS MATERIALS SHOULD BE REMOVED FROM THE PROPOSED PAVED AREAS. THE EXPOSED SUBGRADE SHOULD BE INSPECTED AND APPROVED BY GEOTECHNICAL PERSONNEL AND ANY SOFT AREAS EVIDENT SHOULD BE SUBEXCAVATED AND REPLACED WITH SUITABLE EARTH BORROW APPROVED BY THE GEOTECHNICAL ENGINEER. THE SUBGRADE SHOULD BE SHAPED AND CROWNED TO PROMOTE DRAINAGE OF THE SITE DRAINAGE STRUCTURES. FOLLOWING APPROVAL OF THE PREPARATION OF THE SUBGRADE, THE PAVEMENT GRANULARS MAY BE PLACED.

REFER TO THE GEOTECHNICAL INVESTIGATION PREPARED BY PATERSON DATED NOV 17th, 2021.

No.	REVISIONS	BY	DATE
04	RE-ISSUED FOR APPROVAL	M.L.	06 NOV 2024
03	RE-ISSUED FOR APPROVAL	M.L.	19 JUL 2024
02	RE-ISSUED FOR APPROVAL	M.L.	29 MAY 2024
01	ISSUED FOR APPROVAL	M.L.	08 FEB 2024

NOT AUTHENTIC UNLESS SIGNED AND DATED

LRL ENGINEERING | INGÉNIERIE
5430 Canotek Road | Ottawa, ON, K1J 9G2
www.lrl.ca | (613) 842-3434

CLIENT: LOU FRANGIAN

DESIGNED BY: M.L. DRAWN BY: M.L. APPROVED BY: M.B.

PROJECT: ORLEANS RESIDENTIAL & MEDICAL FACILITY
3996 INNES RD, OTTAWA, ON.

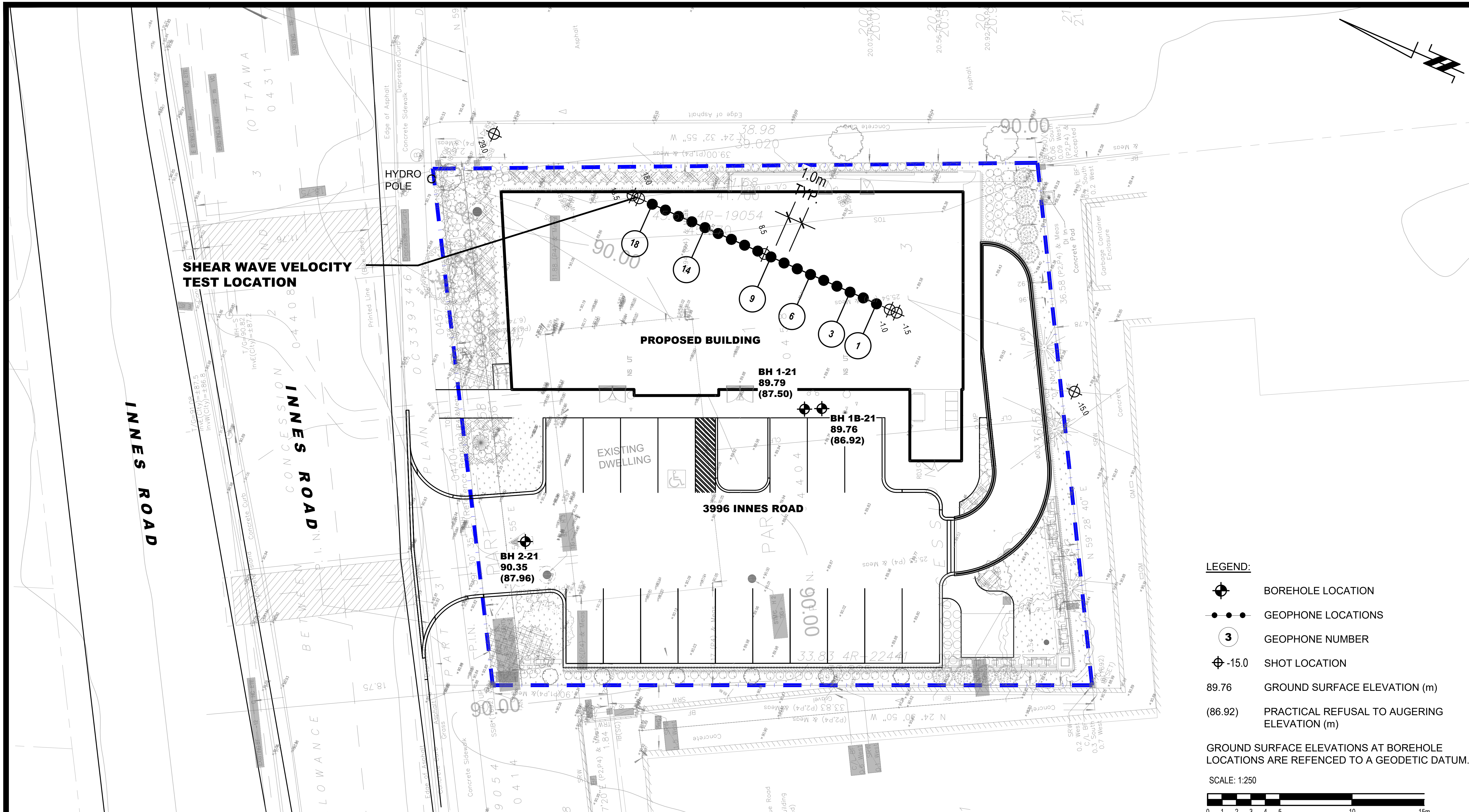
DRAWING TITLE: GRADING AND DRAINAGE PLAN

PROJECT NO. 230737
DATE NOVEMBER 2023

C301

D07-12-21-0209

#18675



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

NO.	REVISIONS	DATE	INITIAL
2	UPDATED TO NEW CONCEPTUAL PLAN	31/01/2025	YZ
1	SEISMIC SHEAR WAVE VELOCITY TEST ADDED TO PLAN	26/05/2022	FC

MR. LOUTFI FRANGIAN

GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
3996 INNES ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 11/2021
Drawn by: YA	Report No.: PG5925-1
Checked by: YZ	Dwg. No.: PG5925-1
Approved by: NP	Revision No.: 2

11x17

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