Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Noise and Vibration Studies

patersongroup

Geotechnical Investigation

Proposed Residential Development
1765 Montreal Road & 9 Beckenham Lane
Ottawa, Ontario

Prepared For

Landric Homes

April 23, 2021

Report: PG5736-1

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Landric Homes to conduct a geotechnical investigation for the proposed residential development to be located at 1765 Montreal Road & 9 Beckenham Lane in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report). The objectives of the geotechnical investigation were to:

❑ Determine the subsoil and groundwater conditions at this site by means of boreholes.

❑ Provide geotechnical recommendations for the 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.

2.0 Proposed Development

Based on the available drawings and correspondence with Landric Homes, it is understood that the proposed development will consist of a multi-storey residential building with up to two underground parking levels as well as low-rise residential houses. Associated landscaped areas, parking areas 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 geotechnical investigation was carried out on April 7, 2021 At that time, 6 boreholes were advanced to a maximum depth of 9.1 m. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the boreholes are shown on Drawing PG5736-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. 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 presented in Appendix 1.

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

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

Diamond drilling was carried out at three boreholes to assess the bedrock depth and quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the Soil Profile and Test Data sheets.

The recovery value is the ratio shown, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one drilled section over the length of the drilled section. These values are indicative of the quality of the bedrock.

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

Groundwater

Flexible polyethylene standpipes were installed within all boreholes to measure the stabilized groundwater levels subsequent to completion of the sampling program. Groundwater conditions were also observed and recorded in the field during the field investigation program.

Sample Storage

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

3.2 Field Survey

The location of the test holes and ground surface elevation at each test hole location were recovered in the field by Paterson personnel. The ground surface elevations at the test hole locations are referenced to a geodetic datum. The test hole locations are presented on Drawing PG5736-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and rock core 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 sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by two existing residential dwellings and associated paved driveways, landscaped areas and underground utilities. The site is bordered by Montreal Road to the south, Beckenham Lane to the west and residential dwellings to the north and east.

The existing ground surface at 1765 Montreal Road is relatively level and at grade with Montreal Road, with the rear yard gradually sloping downward. The existing ground surface at 9 Beckenham Lane is approximately 2 m lower than 1765 Montreal Road at the south property line and slopes downward toward the north. A slope, approximately 2.4 m high, is present along the boundary of the two properties, sloping downward to the north.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of an approximate 0.1 to 0.2 m thick layer of topsoil underlain by fill, extending to a maximum depth of approximately 2.8 m below the existing ground surface. The fill was generally observed to consist of a brown silty sand with crushed stone and gravel with trace to some topsoil and clay.

A thin glacial till deposit was encountered underlying the fill in BH 5-21 at approximate depth of 1.0 m. The glacial till deposit was observed to consist of a brown silty sand with gravel, cobbles and boulders.

Practical refusal to augering was encountered in all boreholes at depths ranging from approximately 1.0 to 1.9 m below the existing ground surface on inferred bedrock, with the exception of borehole BH 2-21 which encountered practical refusal at a depth of 2.8 m below the existing ground surface. However, it should be noted that BH 2-21 was placed within a previous in-ground swimming pool, therefore the inferred bedrock depth at this location is not reflective of the inferred bedrock depth across the remainder of the site.

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

Bedrock

Bedrock was confirmed by coring in boreholes BH 1-21, BH 2-21 and BH 4-21. The bedrock was cored to a maximum depth of 9.1 m below the existing ground surface and consisted of poor to excellent quality grey limestone.

Based on available geological mapping, bedrock in the area of the subject site consists of interbedded limestone from the Gull River Formation with a drift thickness of 0 to 1 m.

4.3 Groundwater

Groundwater levels were measured at flexible polyethylene standpipes installed at all test holes on April 7, 2021. Our groundwater measurements are presented in Table 1 below.

Table 1 - Summary of Groundwater Levels

Borehole Number	Borehole Elevation (m)	Measured Groundwater Level		Recording Date
		Depth (m)	Elevation (m)	
BH 1-21	105.06	1.23	103.83	April 13, 2021
BH 2-21	104.69	2.75	101.94	April 13, 2021
BH 3-21	105.95	Blocked	Blocked	April 13, 2021
BH 4-21	100.58	1.15	99.43	April 13, 2021
BH 5-21	100.69	0.62	100.07	April 13, 2021
BH 6-21	103.16	0.36	102.80	April 13, 2021

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. 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 within 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 level 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 recommended that the proposed building be constructed with conventional footings bearing on the clean, surface sounded bedrock.

The slope observed between the two properties within the subject site are considered to be stable based on our observations.

Bedrock removal will be required for the proposed building excavations. Bedrock removal may also be required for installation of site services, dependent on the depths of the proposed utilities.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant amounts of organic materials or asphalt, should be stripped form under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Bedrock Removal

Where the bedrock is weathered and/or where only small quantities of bedrock need to be removed, hoe ramming is an option for bedrock removal. Where large quantities of bedrock need to be removed, line drilling and controlled blasting may be required. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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 conducted prior to commencing construction.

The extent of the survey should be determined by the blasting consultant and sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocity (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.

Vibration Considerations

Construction operations could be 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 it is caused by blasting operations or by construction operations, could be the cause or the source of detrimental vibrations at the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the permissible vibrations, 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). These guidelines are for current construction standards. 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 minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill used for grading beneath the proposed buildings should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on clean, surface sounded bedrock can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **4,500 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance at ULS.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Footings bearing on an acceptable bedrock bearing surface and designed for 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 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.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. A higher seismic site class, such as Class A, is available. However, a site specific shear wave velocity test is required to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012.

Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab Construction

With the removal of all topsoil and fill, containing deleterious or organic materials, from within the footprint of the proposed buildings, the bedrock will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

For structures with basement slabs, it is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to the minimum 99% of its SPMDD.

5.6 Basement Wall

It is understood that 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 and a 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.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \gamma H$ 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 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 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 $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero. The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of car parking areas and access lanes. It should be noted that for and car only parking areas, an Ontario Traffic Category A is applicable. For access lanes an Ontario Traffic Category B should be used for design purposes.

Table 2 - Recommended Pavement Structure - Car Only Parking	
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, bedrock or OPSS Granular B Type I or II material placed over fill, in situ soil or bedrock.	

Table 3 - Recommended Pavement Structure - Access Lanes/Ramps and Heavy Truck Parking	
Thickness (mm)	**Material Description**
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, bedrock or OPSS Granular B Type I or II material placed over fill, in situ soil or bedrock.	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used. The pavement granular base and subbase should be paced in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

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.

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 structure. It is expected that insufficient room is available for exterior backfill. It is suggested that an adequate drainage system would be as follows:

- Bedrock vertical surface should be prepared to receive the proposed drainage and/or waterproofing system for the entire exposed vertical bedrock excavation. The surface will be prepared by grinding to smooth out angular sections of the bedrock.

- The requirement for a waterproofing membrane, such as a bentonite layer, will be evaluated during the excavation program and will be based on the expected groundwater infiltration volumes. For design purposes, the lower level (P2) should have the bentonite layer included and this layer will be fastened to the bedrock starting from the base of the excavation and extending to the underside of the P1 level.

- A composite drainage layer will be placed against the bentonite membrane (P2 level) and along the entire height of the excavation (P1 level) and will be fastened to the bedrock and shoring system.

- It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to the sump pit(s) within the lower basement area.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 150 mm in perforated pipes be placed in each bay. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Adverse Effects of Dewatering on Adjacent Properties

Since bedrock is relatively shallow at the subject site and the overburden consists mainly of fill and/or a dense glacial till deposit, it's expected that the proposed development will be founded below the long term groundwater level. It's expected that the long term dewatering, resulting from the proposed development, will be negligible.

Any minor dewatering of the site will be within the bedrock layer which is relatively shallow at the subject site. Therefore, no adverse effects to the surrounding buildings or properties are expected with the minor dewatering of the groundwater from this development.

Foundation Backfill

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be relatively frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I, Granular A or Granular B Type II 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 effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard.

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

However, foundations which are founded directly on clean, surface-sounded bedrock, and which is approved by Paterson at the time of construction, is not considered frost susceptible and does not require soil cover.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be 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 expected that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation through bedrock is expected to be completed with near vertical excavation walls. Bedrock stabilization bay be required depending on final depth of excavation. Paterson should review bedrock excavation during construction.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be 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 should be designed by a structural engineer, specializing in shoring design. The shoring will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations, roadways and underground services.

The design and implementation of the temporary systems will be the responsibility of the excavation contractor. The geotechnical information provided below is to assist the contractor in completing a safe shoring system.

The shoring designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event will not negatively impact the shoring system or soils supported by the system. Any changes during construction to the approved shoring design should be reported immediately to the owner's consultants prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. The shoring system could be cantilevered, anchored or braced. Generally, the shoring systems is provided with tie-back rock anchors to ensure the stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. If consideration is given to utilizing a raker style support for the shoring system, the structural engineer should ensure that the design selected minimizes lateral movements to tolerable levels.

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

Table 4 - Soil Parameters

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
Dry Unit Weight (γ), kN/m^3	20
Effective 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 movement is not permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

A 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 & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. The bedding should extend to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of its SPMDD.

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

6.5 Groundwater Control

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

Groundwater Control for Building Construction

A temporary Category 3 Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

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

Adverse Effects from Dewatering on Adjacent Structures

Based on the subsurface conditions encountered at the subject site, it is anticipated that the adjacent structures are founded on bedrock or the dense glacial till deposit. Therefore, no adverse effects from short term and long term dewatering are expected for surrounding structures.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site mostly 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, 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.

The trench excavations should be carried out in a manner that will avoid the introduction of frozen materials into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. In addition, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

The results on analytical testing show that the sulphate content is less than 0.1%. The results are indicative that Type 10 Portland 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 in indicative of a moderate 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.

- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ 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 backfilling.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.
- ❑ Review bedrock excavation activities and exposed vertical bedrock faces.

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.

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.

LICENSED PROFESSIONAL ENGINEER
April 23-2021
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Owen Canton, E.I.T.

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

Report Distribution:

- ❑ Landric Homes (digital copy)
- ❑ Paterson Group

APPENDIX 1

SOIL PROFILE & TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1765 Montreal Road and 9 Beckenham Lane
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE April 7, 2021

FILE NO. PG5736

HOLE NO. BH 1-21

SOIL DESCRIPTION	Depth of change (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	105.06
TOPSOIL	0.08	AU	1				
FILL: Brown silty sand with gravel, trace topsoil	0.61	AU	2				
FILL: Brown silty sand with gravel, crushed stone, cobbles, trace clay		SS	3	54	63	1	104.06
	1.88	SS	4	0	+50	2	103.06
BEDROCK: Good to excellent quality, grey Limestone		RC	1	100	89	3	102.06
		RC	2	100	97	4	101.06
		RC	3	100	100	5	100.06
		RC	4	100	100	6	99.06
						7	98.06
	8.53	RC	5	100	100	8	97.06
End of Borehole							
(GWL @ 1.23m - April 13, 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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1765 Montreal Road and 9 Beckenham Lane
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE April 7, 2021

FILE NO. PG5736

HOLE NO. BH 2-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	104.69
Asphaltic Concrete	0.03	AU	1				
FILL: Brown silty sand with crushed stone	0.10						
Asphaltic Concrete	0.15	AU	2				
FILL: Brown silty sand with crushed stone	0.64	SS	3	46	4	1	103.69
FILL: Brown silty sand, some gravel, trace clay	2.84	SS	4	17	2	2	102.69
		SS	5	21	8		
BEDROCK: Fair to excellent quality, grey limestone	8.41	RC	1	100	71	3	101.69
						4	100.69
		RC	2	100	67	5	99.69
						6	98.69
		RC	3	100	100	7	97.69
		RC	4	100	100	8	96.69
End of Borehole							
(GWL @ 2.75m - April 13, 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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1765 Montreal Road and 9 Beckenham Lane
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE April 7, 2021

FILE NO. PG5736

HOLE NO. BH 3-21

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 %	Piezometer Construction
GROUND SURFACE						0	105.95		
TOPSOIL 0.18		AU	1						
FILL: Brown silty sand with crushed stone, trace cobbles 0.69		AU	2						
BEDROCK: Poor quality, grey limestone 1.04						1	104.95		
End of Borehole									
Practical refusal to augering at 1.04m depth									
(BH dry upon completion)									

20	40	60	80	100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1765 Montreal Road and 9 Beckenham Lane
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** April 7, 2021

FILE NO. PG5736

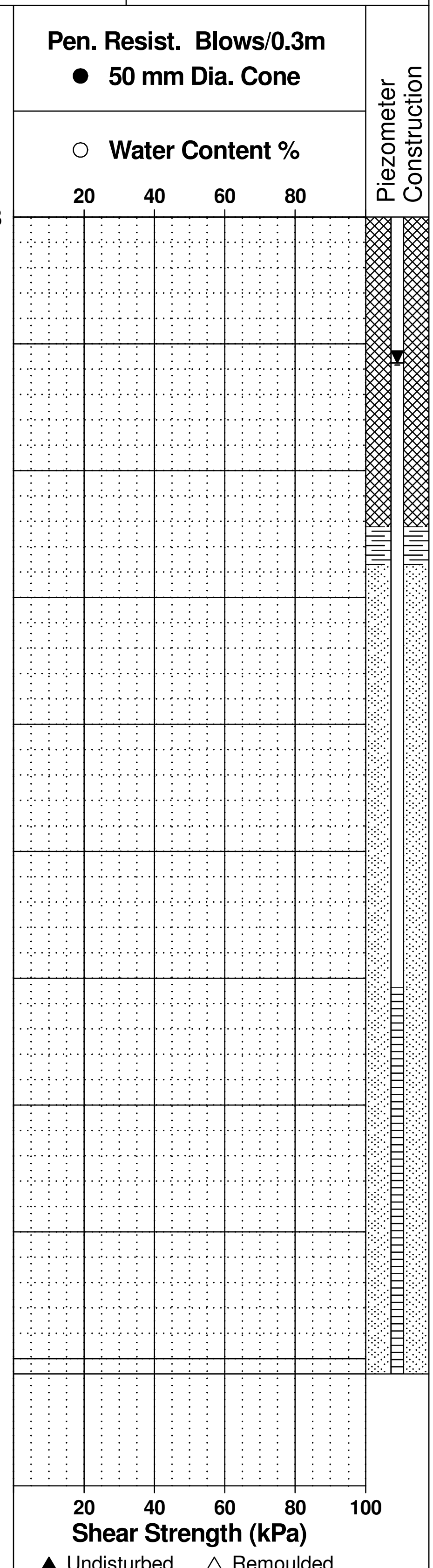
HOLE NO. BH 4-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.20	AU	1		
FILL: Brown silty sand trace clay and organics	0.69	AU	2		
FILL: Brown silty sand	0.91				
FILL: Brown silty sand trace topsoil	1.27	SS	3	100	+50
BEDROCK: Poor to excellent quality, grey limestone	9.12	RC	1	100	0
		RC	2	100	46
		RC	3	97	87
		RC	4	100	73
		RC	5	98	80
		RC	6	100	100

End of Borehole

(GWL @ 1.15m - April 13, 2021)

DEPTH (m)	ELEV. (m)
0	100.58
1	99.58
2	98.58
3	97.58
4	96.58
5	95.58
6	94.58
7	93.58
8	92.58
9	91.58



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1765 Montreal Road and 9 Beckenham Lane
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE April 7, 2021

FILE NO. PG5736

HOLE NO. BH 5-21

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	100.69		
TOPSOIL 0.10		AU	1						
FILL: Brown silty sand with topsoil, organics and gravel 0.51		AU	2						
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders 0.97									
BEDROCK: Good quality grey limestone 1.50		SS	3	25	+50	1	99.69		
End of Borehole									
(Piezometer blocked at 0.62m depth - April 13, 2021)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1765 Montreal Road and 9 Beckenham Lane
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE April 7, 2021

FILE NO. PG5736

HOLE NO. BH 6-21

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	103.16		
TOPSOIL 0.15		AU	1						
FILL: Brown silty sand with topsoil, organics, trace clay and gravel 0.71		AU	2						
BEDROCK: Fragmented grey limestone 1.40		SS	3	38	+50	1	102.16		
End of Borehole									
Practical refusal to augering at 1.40m depth									
(Piezometer blocked at 0.73m depth - April 13, 2021)									

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

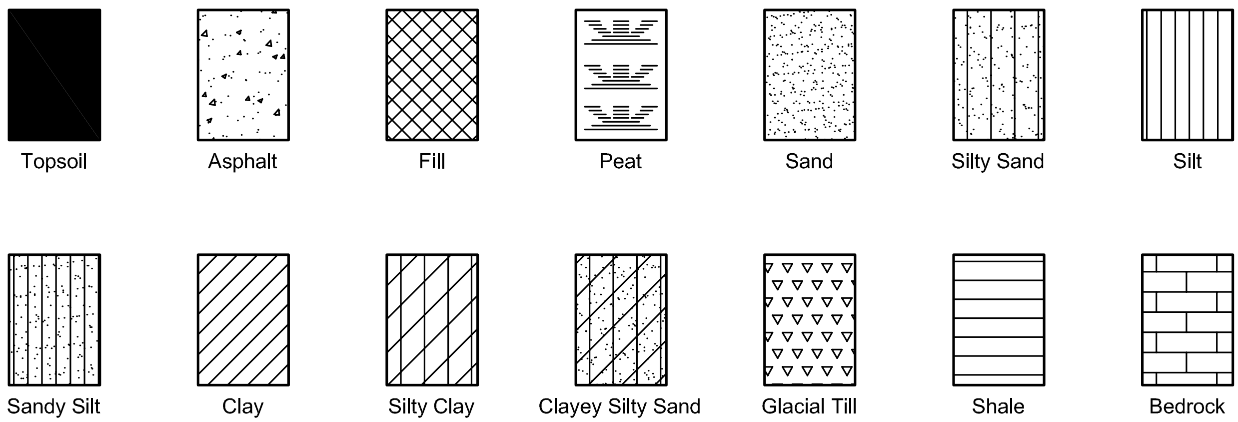
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

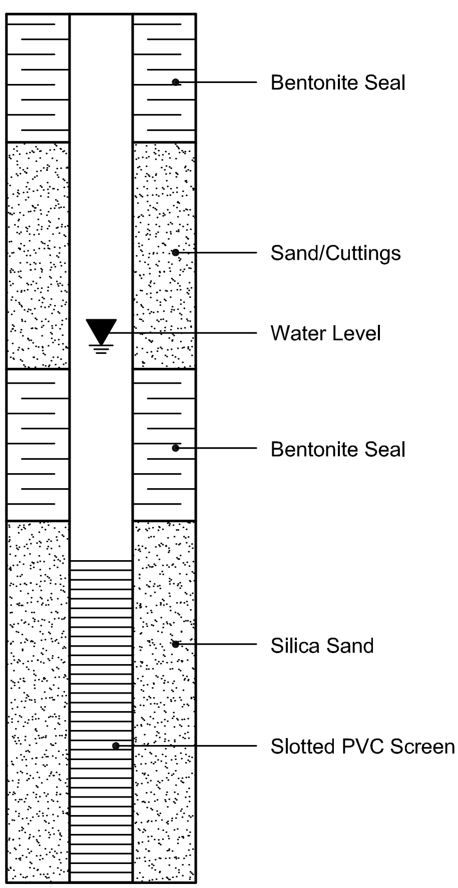
SYMBOLS AND TERMS (continued)

STRATA PLOT

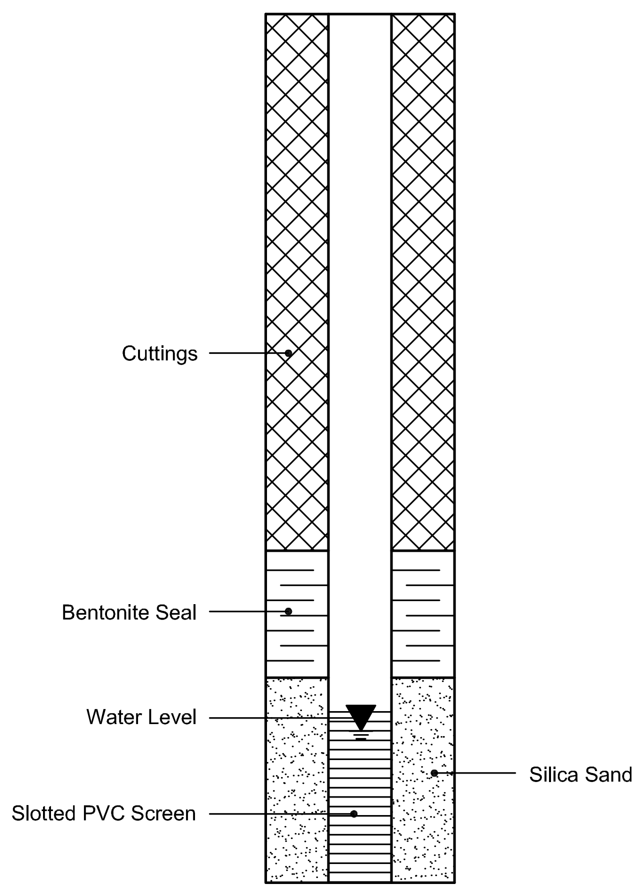


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

Order #: 2115642

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29752

Report Date: 14-Apr-2021

Order Date: 9-Apr-2021

Project Description: PG5736

	Client ID:	BH2-21 SS5	-	-	-
	Sample Date:	06-Apr-21 09:00	-	-	-
	Sample ID:	2115642-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	90.8	-	-	-
General Inorganics					
pH	0.05 pH Units	7.94	-	-	-
Resistivity	0.10 Ohm.m	47.5	-	-	-
Anions					
Chloride	5 ug/g dry	8	-	-	-
Sulphate	5 ug/g dry	120	-	-	-

APPENDIX 2

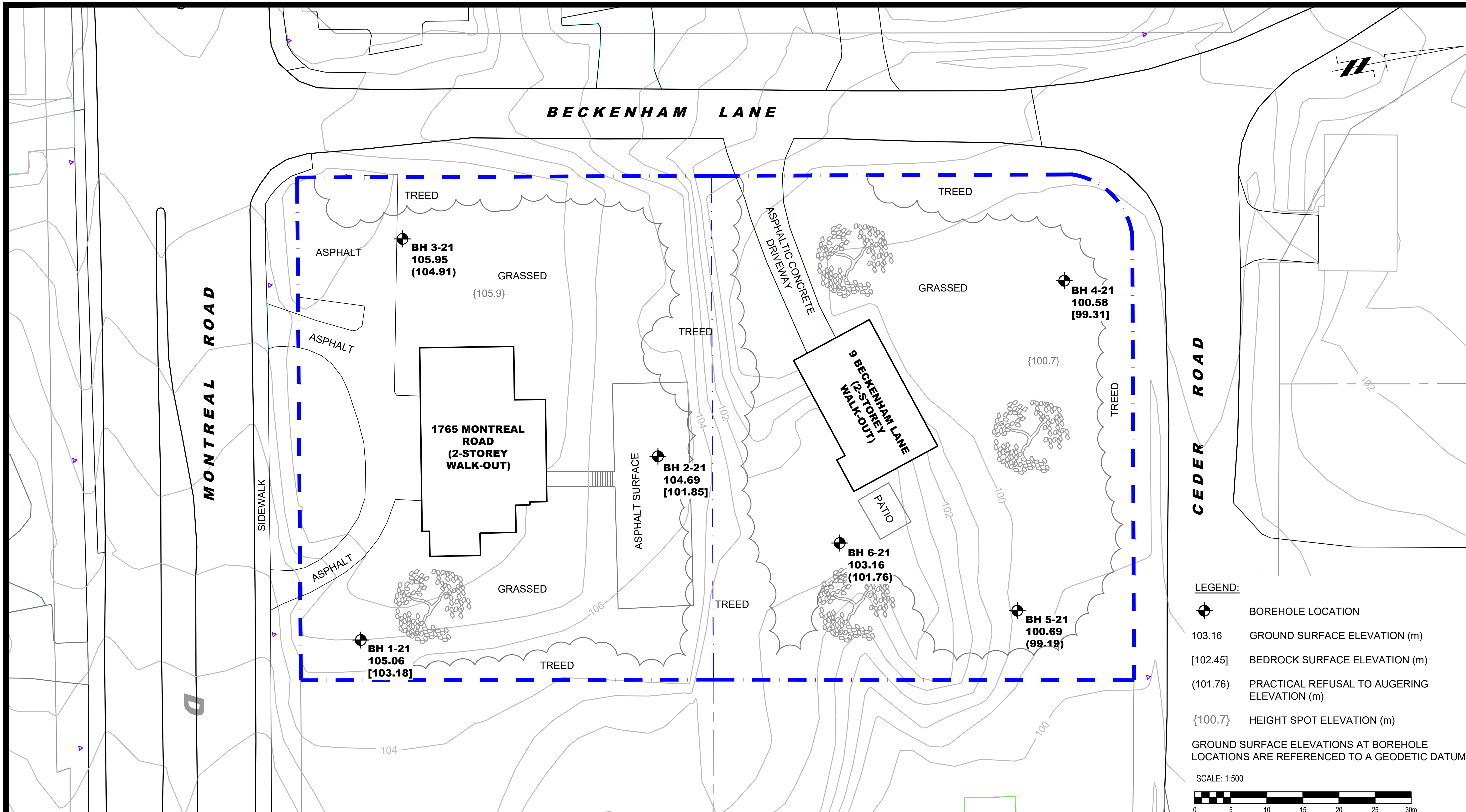
FIGURE 1 - KEY PLAN

DRAWING PG5736-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

patersongroup



patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

LANDRIC HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
1765 MONTREAL ROAD AND 9 BECKENHAM LANE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 04/2021
Drawn by: YA	Report No.: PG5736-1
Checked by: OC	Dwg. No.: PG5736-1
Approved by: DJG	Revision No.:

p:\autocad drawings\geotechnical\pg57xx\pg5736\pg5736-1-test hole location plan.dwg