PATERSON GROUP

Geotechnical Investigation

Proposed Residential Development

1883 Stittsville Main Street
Ottawa, Ontario

Prepared for Mattamy Homes

Report PG7178-1 dated July 2, 2024

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Mattamy Homes to conduct a geotechnical investigation for the proposed residential development to be located at 1883 Stittsville Main Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 for the general site location).

The objectives of the geotechnical investigation were to:

- ❑ Determine the subsoil and groundwater conditions at this site by means of test pits.
- ❑ Provide geotechnical recommendations pertaining to 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.

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

2.0 Proposed Development

Based on the available conceptual drawings, it is understood that the proposed development will consist of back-to-back style townhouses which may include up to 1 basement level.

At finished grades, the proposed buildings will generally be surrounded by asphalt-paved access lanes, parking areas, and walkways with landscaped margins. It is also understood that the proposed development is to be municipally serviced.

It is expected that the existing residential dwelling will need to be demolished to accommodate the construction of the proposed buildings.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on June 18, 2024, and consisted of advancing a total of 6 test pits to a maximum depth of 3.0 m below the existing ground surface. The test pit locations were distributed in a manner to provide general coverage of the proposed development, taking into consideration existing site features and underground services. The approximate locations of the test pits are shown on Drawing PG7178-1 - Test Hole Location Plan included in Appendix 2.

The test pits were excavated using a backhoe and backfilled with the excavated soil upon completion. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The test pitting procedure consisted of excavating to the required depth at the selected locations, and sampling and testing the overburden.

Sampling and In-Situ Testing

Soil samples were recovered from the sidewalls of the test pits. All soil samples were visually inspected and classified on site. The soil samples were placed in sealed plastic bags and transported to our laboratory for further examination and classification. The depths at which the soil samples were recovered from the test pits are shown as G on the Soil Profile and Test Data sheets presented in Appendix 1.

The subsurface conditions observed in the test pits 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

Where present, groundwater infiltration levels and soil color changes were recorded in all test pits prior to backfilling at the time of excavation of the test pits. The groundwater observations are further discussed in Section 4.3 and are presented in the Soil Profile and Test Data Sheets in Appendix 1.

3.2 Field Survey

The test pit locations, and the ground surface elevations at each test pit location, were surveyed by Paterson using a handheld GPS unit, and referenced to a geodetic datum. The test pit locations are presented on Drawing PG7178-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were collected during the excavation of the test pits, and visually examined in our laboratory to review the results of the field logging. 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.

4.0 Observations

4.1 Surface Conditions

The northern portion of the subject site is currently occupied by an existing residential dwelling and asphalt-paved driveway. The southern portion of the site generally consists of landscaped areas with mature trees.

The subject site is bordered to the west by Stittsville Main Street, to the north by Parade Drive, to the east by Falabella Street, and to the south by Campolina Way. The site generally slopes downward from west to east, from approximate geodetic elevation 122 to 119 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the subject site consists of topsoil underlain by a glacial till deposit. At test pits TP 1-24 and TP 3-24, a fill layer was observed underlying the topsoil material and extended to approximate depths of 0.3 and 0.1 m, respectively. The fill was observed to consist of loose, brown silty sand with topsoil and organics as well as trace gravel and cobbles.

A deposit of glacial till was observed underlying the topsoil and/or fill layer at all test pit locations. The glacial till deposit consisted of compact to dense brown silty sand to sandy silt with gravel, cobbles and boulders and extended to the bedrock surface.

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

Bedrock

Weathered bedrock was encountered at depths ranging from 0.9 to 2.7 m, generally increasing in depth from west to east across the site. The weathered bedrock was observed to consist of black shale. Practical refusal to excavation on competent bedrock was encountered in all test pits at approximate depths ranging from 1.4 to 3.0 m below the existing ground surface.

4.3 Groundwater

Groundwater infiltration into the open test pits was not observed at the time of the field program. However, long-term groundwater levels can also be estimated based on the observed colour, moisture content and consistency of the recovered soil samples. Based on these observations, it is anticipated that the groundwater table is located within the bedrock.

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 recommended that the proposed residential buildings be founded on conventional spread footings bearing either on the compact to dense glacial till, undisturbed weathered bedrock, and/or clean surface sounded bedrock.

Expansive shale bedrock may present at this site. Precautions should be provided during construction to reduce the risks associated with the potentially heaving shale bedrock, which are discussed further in Section 6.7.

Due to relatively shallow bedrock depth across the site, it is anticipated that bedrock removal will be required for building construction and site servicing. All contractors should be prepared for bedrock removal within the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and any fill containing significant amounts of deleterious or organic materials, should be stripped from under any buildings and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities.

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

Bedrock Removal

In areas where shallow bedrock is encountered, and where the bedrock is weathered and only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming. However, dependent on the quantity and condition of the bedrock, line-drilling in conjunction with hoe-ramming may be required to remove the bedrock. Sound bedrock may be removed by line drilling in conjunction with 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 the proximity of the blasting operations should be carried out 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 or claims related to the blasting operations.

The blasting operations must be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Vibration Considerations

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

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations 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).

It should be noted that these guidelines are for today's construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Fill Placement

Engineered fill used for grading beneath the proposed building 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. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the propose building should be compacted to a minimum 98% of the 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. These materials 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.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm. Where this fill material is open-graded, a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. Site-generated blast rock fill should be compacted using a suitably sized smooth drum vibratory roller when considered for placement. This can be assessed at the time of construction.

Under winter conditions, if snow and ice is present within the blast rock fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. The geotechnical consultant should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized.

Lean Concrete Filled Trenches

As discussed above, where the clean, surface sounded bedrock is encountered below the underside of footing (USF) elevation, zero-entry vertical trenches should be excavated and backfilled with lean concrete (minimum **17 MPa** 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The lean concrete placement should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured will suffice in providing a direct transfer of the footing load to the underlying clean, surface sounded bedrock. Once the trench excavation is approved by Paterson, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on an undisturbed, weathered bedrock, or compact to dense glacial till can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa.** A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

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

Footings designed using the bearing resistance value at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Footings supported directly on clean, surface-sounded bedrock, or on lean concrete which is placed directly over the clean surface-sounded bedrock, can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,000 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value 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 supported directly on clean, surface sounded bedrock and design for the bearing resistance values provided above will be subject to negligible 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. Above the groundwater level, adequate lateral support is provided to the in-situ bearing medium soils when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A heavily fractured, weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Bedrock/Soil Transition

Where a footing is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the portion placed on a soil bearing medium to reduce the potential long-term total and differential settlements.

Also, at the soil/bedrock transitions, it is recommended that a minimum depth of 500 mm of bedrock be removed from below the founding elevation for a minimum length of 2 m on the bedrock side. This area should be subsequently reinstated with an engineered fill, such as OPSS Granular A or Granular B Type II and compacted to a minimum of 98% of the material SPMDD.

The width of the sub-excavation should be a minimum of 500 mm greater than the width of the footing. Steel reinforcement, extending a minimum of 3 m on both sides of the 2 m long transition, should be placed in the top portions of the footing and foundation walls.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. If a higher seismic site class is required (Class A or B) for the proposed residential buildings, and the proposed footings are to be located within 3 m of the bedrock surface, a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as defined 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 version of the OBC 2012 for a full discussion of the earthquake design requirements.

5.5 Floor Slab Construction

With the removal of all topsoil and deleterious fill from within the footprint of the proposed building, the glacial till or bedrock medium will be considered acceptable subgrades on which to commence backfilling for floor slab construction.

For structures with slab-on-grade construction, it is recommended that the upper 200 mm of sub-slab fill consist of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed structures should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the material's SPMDD.

If a basement level is considered for the proposed building, it is recommended that the upper 300 mm of sub-floor fill consists of 19 mm clear crush stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the material's SPMDD.

5.6 Pavement Design

Car only parking areas, access lanes and heavy truck parking/loading areas are anticipated at this site. For the proposed surface parking areas, the pavement structures provided in Tables 1 and 2 are recommended.

Table 1 - Recommended Asphalt Pavement Structure - Car Only Parking Areas	
Thickness (mm)	**Material Description**
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE – Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill.	

Table 2 - Recommended Asphalt Pavement Structure - Access Lanes and Heavy Loading Parking Areas	
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
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 or fill.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable compaction equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage & Waterproofing

Foundation Drainage

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

Backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent thickness of soil cover and foundation insulation, should be provided in this regard.

Exterior unheated foundations, such as isolated piers, are more prone to deleterious movement associated with frost action than the exterior walls of the 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 with no cracks or fissures, 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 overburden and weathered bedrock 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. For the proposed development, it is anticipated that sufficient room will be available for the greater part of the excavations to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes in the overburden soils and weathered bedrock, above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. Excavations below the groundwater level should be cut back at a maximum slope of 1.5H:1V. 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.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system. Where sufficient space for the horizontal ledge is not available, it is recommended that concrete blocks be used to retain the overburden soils.

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 a soil or weathered bedrock subgrade. If the bedding is placed on clean, surface sounded 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. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe, should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 95% of the SPMDD.

It should generally be possible to re-use the site generated fill materials (moist, not wet) above the cover material if excavation and filling operations are carried out in dry and non-freezing weather conditions. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

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) and above the cover material 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 excavation 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.

Permit to Take Water

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

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

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 using 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 into 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 Protection of Potentially Expansive Shale Bedrock

Upon being exposed to air and moisture, shale may decompose into thin flakes along the bedding planes. Previous studies have concluded shales containing pyrite are subject to volume changes upon exposure to air. As a result, the formation of jarosite crystals by aerobic bacteria occurs under certain ambient conditions.

It has been determined that the expansion process does not occur or can be retarded when air (i.e. oxygen) is prevented from contact with the shale and/or the ambient temperature is maintained below 20ºC, and/or the shale is confined by pressures in excess of 70 kPa. The latter restriction on the heaving process is probably the major reason why damage to structures has, for the greater part, been confined to slabs-on-grade rather than footings.

Based on the test pit logs, expansive shale may be encountered at the subject site. To reduce the long term deterioration of the shale, exposure of the bedrock surface to oxygen should be kept as low as possible.

The weathered bedrock surface within the proposed building footprint should be protected from excessive dewatering and exposure to ambient air. A 50 mm thick concrete mud slab, consisting of minimum 17 MPa lean concrete, should be placed on the exposed bedrock surface within a 48 hour period of being exposed. The excavated sides of the exposed bedrock should be sprayed with shotcrete to seal bedrock from exposure to air and dewatering.

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.
- ❑ 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.***

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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 Mattamy 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.

Mrunmayi Anvekar, M.Eng.

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Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1883 Stittsville Main Street
Ottawa, Ontario

EASTING: 350930.109 **NORTHING:** 5011425.879 **ELEVATION:** 124.43

DATUM: Geodetic

REMARKS:

BORINGS BY: Backhoe **DATE:** June 18, 2024

FILE NO. PG7178

HOLE NO. TP 1-24

SAMPLE DESCRIPTION	Depth to base (m)	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	124.43
TOPSOIL and organics	0.05						
FILL: Loose brown silty sand, trace gravel and cobbles	0.30	G	1				
GLACIAL TILL: Compact brown silty sand, trace gravel, cobbles	0.90	G	2				
BEDROCK: Weathered black shale bedrock	2.00					1	123.43
End of Test Pit Practical refusal to excavation at 2.0 m						2	122.43

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

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1883 Stittsville Main Street
Ottawa, Ontario

EASTING: 350952.898 **NORTHING:** 5011402.061 **ELEVATION:** 122.62

DATUM: Geodetic

REMARKS:

BORINGS BY: Backhoe **DATE:** June 18, 2024

FILE NO. PG7178

HOLE NO. TP 2-24

SAMPLE 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	122.62		
TOPSOIL and organics — 0.10									
GLACIAL TILL: Compact to dense brown silty sand with gravel and cobbles - Dense and with boulders by 0.8 m		G	1						
— 1.00						1	121.62		
BEDROCK: Weathered black shale bedrock		G	2						
— 1.40									
End of Test Pit Practical refusal to excavation at 1.4 m									

Water Content % axis: 20 40 60 80

Shear Strength (kPa) axis: 20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1883 Stittsville Main Street
Ottawa, Ontario

EASTING: 350953.945 **NORTHING:** 5011349.338 **ELEVATION:** 124.02

DATUM: Geodetic

REMARKS:

BORINGS BY: Backhoe **DATE:** June 18, 2024

FILE NO. PG7178

HOLE NO. TP 3-24

SAMPLE DESCRIPTION	Depth to base of layer (m)	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	124.02
FILL: Loose brown silty sand with topsoil and organics	0.10						
GLACIAL TILL: Dense brown silty sand with gravel, cobbles and boulders		G	1				
		G	2			1	123.02
		G	3				
						2	122.02
		G	4				
	2.70	G	5				
BEDROCK: Weathered black shale bedrock	3.00					3	121.02
End of Test Pit Practical refusal to excavation at 3.0 m							

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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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1883 Stittsville Main Street
Ottawa, Ontario

EASTING: 350990.031 **NORTHING:** 5011384.755 **ELEVATION:** 121.75

DATUM: Geodetic

REMARKS:

BORINGS BY: Backhoe **DATE:** June 18, 2024

FILE NO. PG7178

HOLE NO. TP 4-24

SAMPLE 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									
TOPSOIL and organics						0	121.75		
0.20									
GLACIAL TILL: Compact brown silty sand with gravel and cobbles		G	1						
						1	120.75		
		G	2						
1.25									
GLACIAL TILL: Compact brown silty sand to sandy silt, with gravel, cobbles and boulders		G	3						
2.00						2	119.75		
BEDROCK: Weathered bedrock									
2.50									
End of Test Pit Practical refusal to excavation at 2.5 m									

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1883 Stittsville Main Street
Ottawa, Ontario

EASTING: 351000.559 **NORTHING:** 5011412.837 **ELEVATION:** 121.30

DATUM: Geodetic

REMARKS:

BORINGS BY: Backhoe **DATE:** June 18, 2024

FILE NO. PG7178

HOLE NO. TP 5-24

SAMPLE 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	121.30		
TOPSOIL and organics with brown silty sand, trace gravel (to 0.30)									
GLACIAL TILL: Compact brown silty sand with gravel and cobbles, trace boulders		G	1						
		G	2						
(to 1.10)						1	120.30		
GLACIAL TILL: Compact brown silty sand to sandy silt with gravel, cobbles and boulders		G	3						
(to 1.60)									
End of Test Pit Practical refusal to excavation at 1.6 m									

Water Content % axis: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1883 Stittsville Main Street
Ottawa, Ontario

EASTING: 350975.905 **NORTHING:** 5011394.776 **ELEVATION:** 121.76

DATUM: Geodetic

REMARKS:

BORINGS BY: Backhoe **DATE:** June 18, 2024

FILE NO. PG7178

HOLE NO. TP 6-24

SAMPLE 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								20 40 60 80	
TOPSOIL and organics — 0.20						0	121.76		
GLACIAL TILL: Compact silty sand with gravel, cobbles and shale fragments		G	1						
— 1.05		G	2			1	120.76		
BEDROCK: Weathered black shale bedrock — 1.35									
End of Test Pit Practical refusal to excavation at 1.35 m									

Shear Strength (kPa): 20 40 60 80 100

▲ 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 strength of cohesionless soils is the relative density, 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.

Relative Density	‘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 vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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 is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

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 NXL size core. However, it can be used on smaller core sizes, such as BX, 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
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture 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 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 x 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

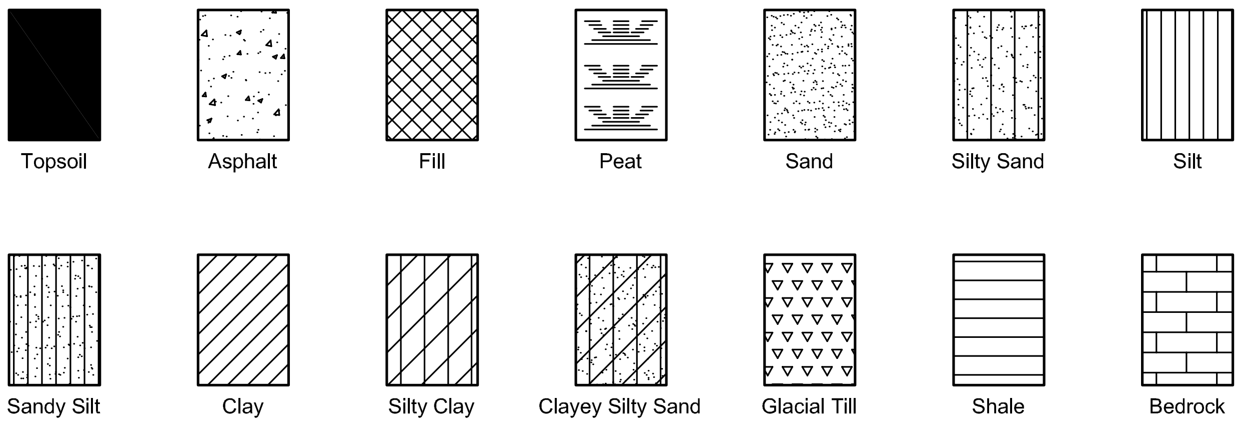
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

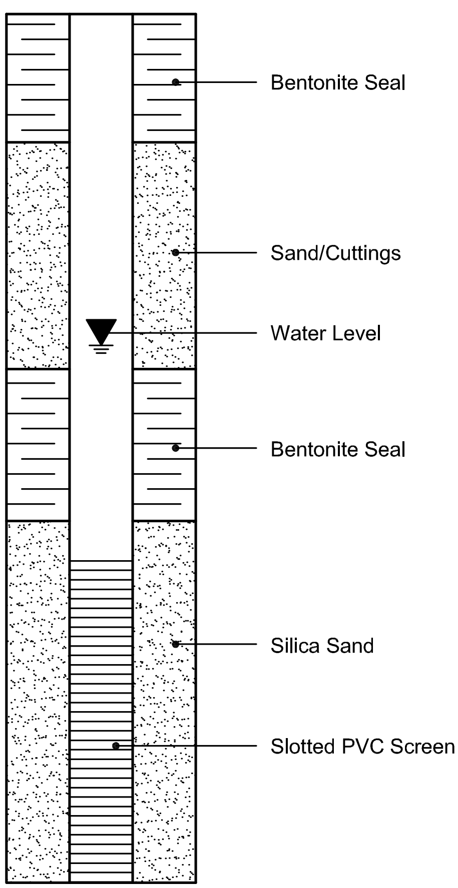
SYMBOLS AND TERMS (continued)

STRATA PLOT

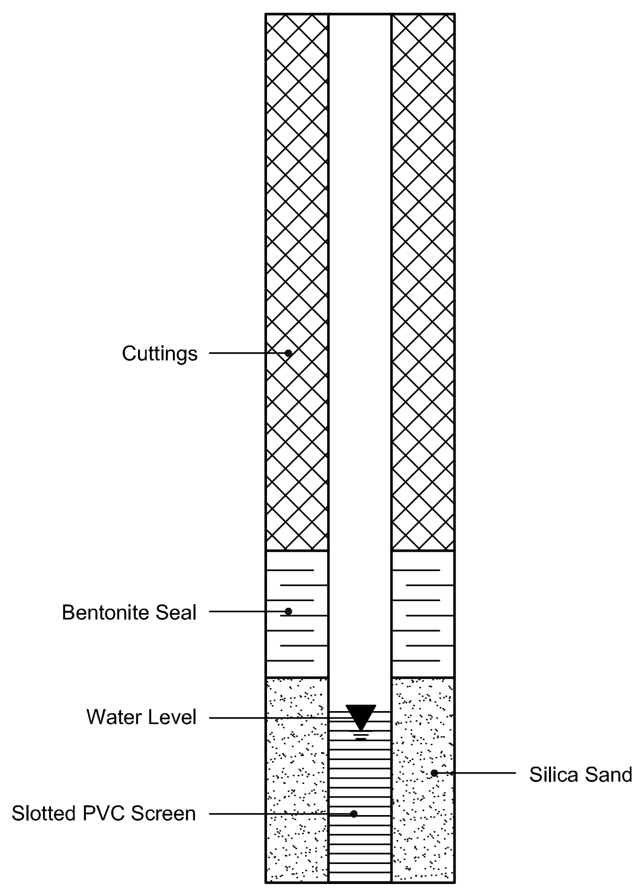


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG7178-1 - TEST HOLE LOCATION PLAN

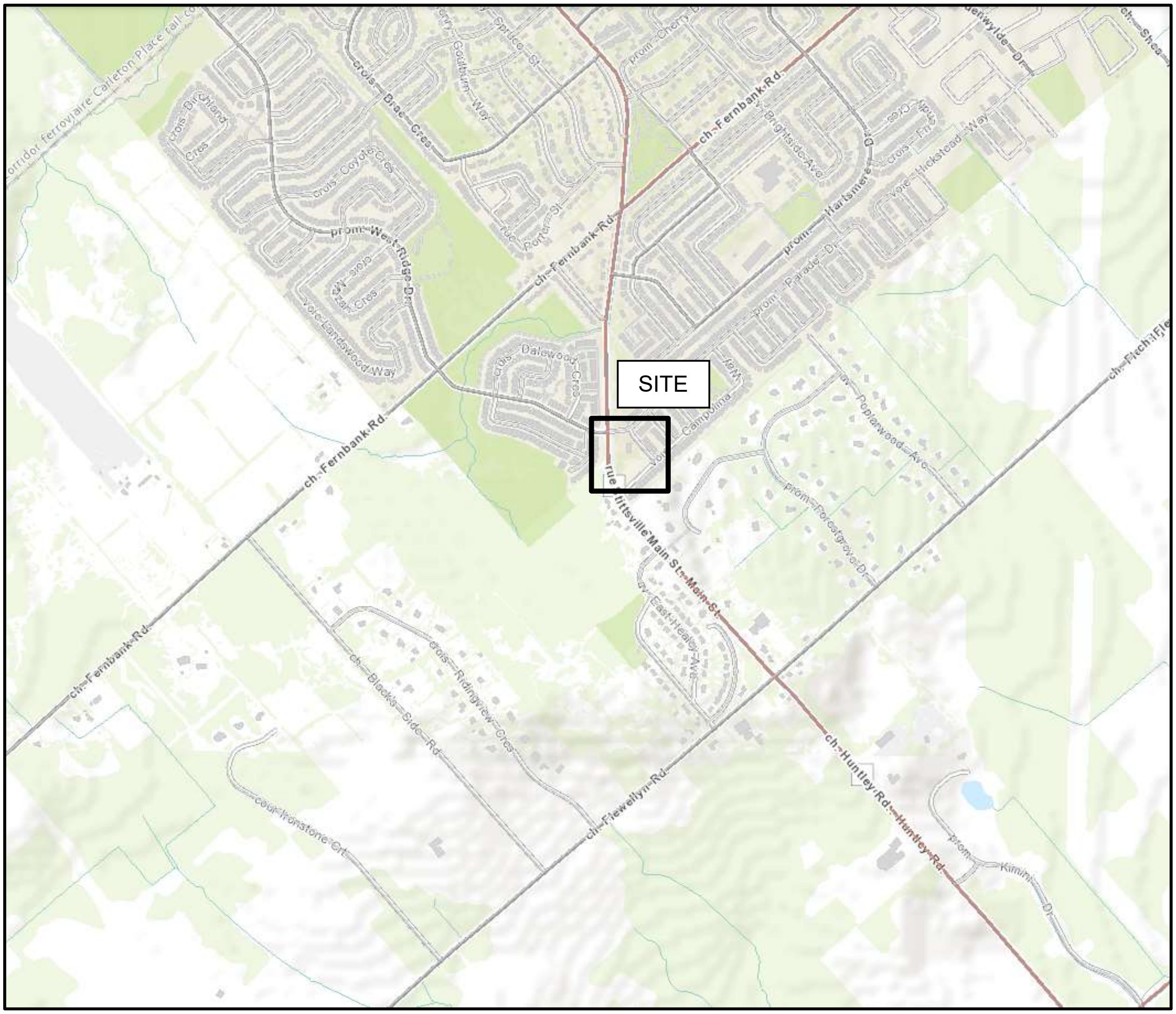
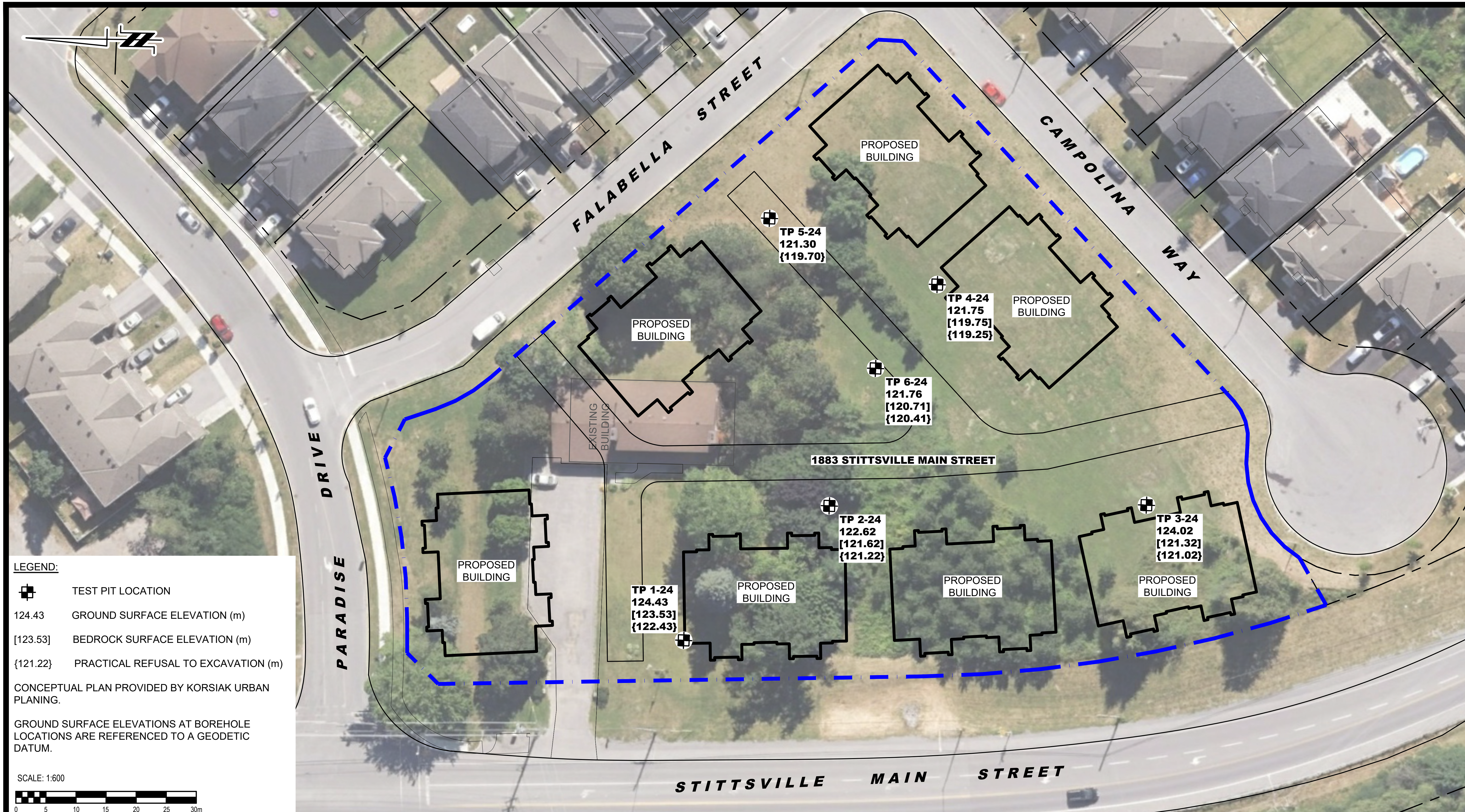


FIGURE 1

KEY PLAN

PATERSON GROUP



LEGEND:

(symbol)	TEST PIT LOCATION
124.43	GROUND SURFACE ELEVATION (m)
[123.53]	BEDROCK SURFACE ELEVATION (m)
{121.22}	PRACTICAL REFUSAL TO EXCAVATION (m)

CONCEPTUAL PLAN PROVIDED BY KORSIAK URBAN PLANING.

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

SCALE: 1:600

0 5 10 15 20 25 30m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

MATTAMY HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
1883 STITTSVILLE MAIN STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:600	Date: 06/2024
Drawn by: GK	Report No.: PG7178-1
Checked by: MA	Dwg. No.: PG7178-1
Approved by: KP	Revision No.:

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