Geotechnical Investigation

Proposed Multi-Storey Building

1773 and 1767 Baseline Road
Ottawa, Ontario

Prepared for Mr. Shiv Bhasker c/o DCR Phoenix Homes

Report PG7446-1 dated June 19, 2025

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Mr. Shiv Bhasker c/o DCR Phoenix Homes to conduct a geotechnical investigation for the proposed multi-storey building to be located at 1773 and 1767 Baseline Road in the City of Ottawa (reference should be made to Figure 1 – Key Plan in Appendix 2 of this report 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 boreholes.

- 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.

2.0 Proposed Development

Based on the available conceptual plan, it is understood that the proposed development will consist of a five-storey residential building with one underground parking level, occupying the majority of the subject site.

Further, it is understood that the remainder of the site will generally be occupied by access lanes, amenity areas, and landscaped areas. It is also expected that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on February 18, 2025, and consisted of advancing a total of 2 boreholes to a maximum depth of 7.1 m below the existing ground surface.

The borehole locations were determined in the field by Paterson personnel and distributed in a manner to provide general coverage of the subject site taking into consideration existing site features and underground utilities. The test hole locations are presented on Drawing PG7446-1 – Test Hole Location Plan included in Appendix 2.

The boreholes were advanced with a low-clearance track-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The drilling procedure consisted of augering to the required depth at the selected location, sampling, and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered from the boreholes using a 50 mm diameter split-spoon sampler or from the auger flights. The split spoon, and auger samples were classified on site and placed in sealed plastic bags. All soil samples were transported to our laboratory for further review. The depths at which the split spoon, and auger samples were recovered from the boreholes are shown as SS, and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus in all boreholes.

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

Groundwater

All boreholes were fitted with a flexible polyethylene standpipe to allow groundwater level monitoring. The groundwater level readings were obtained after a suitable stabilization period subsequent to the completion of the field investigation.

The groundwater observations are discussed in Section 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

All samples from the current investigation will be stored in the laboratory for a period of 1 month after the issuance of this report. They will then be discarded unless directed otherwise.

3.2 Field Survey

The borehole locations and ground surface elevation at each borehole location were surveyed by Paterson using a high-precision handheld GPS and referenced to a geodetic datum. The locations of the boreholes, and the ground surface elevation at each borehole location, are presented on Drawing PG7334-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by two residential dwellings located centrally within the site and facing Baseline Road, with the remainder of the site being occupied by asphalt-surfaced parking areas, landscaped areas, and mature trees.

The site is bordered by residential dwellings to the north and east, by an apartment building to the west, and by Baseline Road to the south. The ground surface elevation across the subject site is relatively flat and at-grade with surrounding properties and streets at approximate geodetic elevation of 88 m.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the borehole locations consists of a fill layer extending to a maximum depth of approximately 2.2 m below the existing ground surface underlain by a silty clay deposit. The fill was generally observed to consist of brown silty clay with sand and gravel, extending to approximately 0.3 m below existing grade. The remainder of the fill layer generally consists of brown silty clay with trace gravel.

The silty clay layer was generally observed to consist of very stiff to stiff, brown silty clay crust underlain by stiff to firm grey silty clay at an approximate depth of 3.7 m below the existing ground surface.

A glacial till deposit was encountered underlying the clay at an approximate depth of 5.3 m below the existing ground surface. The glacial till was observed to consist of compact ,grey silty clay with gravel, cobbles, and boulders.

Practical refusal to augering was encountered at depths of 7.1 m and 6.6 m below the existing ground surface at boreholes BH 1-25 and BH 2-25, respectively.

Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Based on available geological mapping, the bedrock in the subject area is part of the Gull River formation, which consists of interbedded limestone and dolomite with an overburden drift thickness ranging between 5 to 10 m.

4.3 Groundwater

Groundwater levels were recorded at each borehole locations and are presented in Table 1 below. The groundwater level readings are also presented in the Soil Profile and Test Data sheets in Appendix 1.

Table 1 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1-25	87.84	4.40	83.44	February 28, 2025
BH 2-25	87.92	4.55	83.37	

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

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. The long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples.

Based on these observations, the long-term groundwater table can be expected at a depth of approximately **4 to 5 m** below the existing ground surface.

However, groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. Given the anticipated structural loads, it is expected that conventional spread footings bearing on the firm silty clay will not provide sufficient bearing for support of the proposed building. Accordingly, it is recommended that foundation support for the proposed building consist of one of the following:

- a raft foundation placed on the undisturbed, firm to stiff grey silty clay, or

- conventional spread footings bearing on lean concrete filled trenches which extend to the undisturbed, compact glacial till.

Due to the presence of a silty clay layer, the proposed development will be subjected to a permissible grade restriction. If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term total and differential settlements.

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

5.2 Grading and Preparation

Stripping Depth

Asphalt, topsoil, construction debris, and deleterious fill, such as those containing organic materials, should be removed from within the perimeter of the proposed building and from under paved areas, pipe bedding or other settlement sensitive structures.

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

Fill Placement

Fill placed for grading beneath the proposed building should consist, unless otherwise specified, of clean imported granular fill such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the proposed building should be compacted to a minimum of 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified site-excavated soil could be placed as general landscaping fill and beneath paved areas. In landscaped areas, these materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. In areas to be paved, the site-excavated soils should be compacted to a minimum of 98% of the material's standard Proctor maximum dry density (SPMDD).

Vibration Considerations

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

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

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline for construction activities, the peak particle velocity should be less than 19 mm/s between frequencies of 4 to 15 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 15 and 40 Hz).

5.3 Foundation Design

Conventional Spread Footings

Conventional spread footings supported on lean concrete filled trenches extending to the undisturbed, compact glacial till can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the above-noted bearing resistance values at ULS.

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

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

Lean Concrete Filled Trenches

Typically, the excavation side walls will be used as the form to support the lean concrete. The trench excavation should be at least 150 mm wider than all sides of the footings. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying glacial till layer. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation. The trenches should be infilled with lean concrete (minimum 17 MPa, 28-day strength).

Raft Foundation

For this proposed building with one underground level, consideration can be provided to a raft foundation if the building loads are not acceptable for conventional spread footings placed over a silty clay bearing surface. Based on available data, the following parameters should be considered for raft design.

For design purposes, the raft foundation base is anticipated to be located at a depth of about 4 m to accommodate one level of underground parking. The bearing medium will consist of firm, grey silty clay which is susceptible to disturbance under construction traffic. It is recommended that a minimum 50 mm thick mud slab be placed immediately after exposure, in order to protect the bearing surface from disturbance.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The bearing resistance value at SLS (contact pressure) of **125 kPa** will be considered acceptable. The loading conditions for the contact pressure are based on sustained loads, generally considered to be 100% Dead Load and 50% Live Load.

The factored bearing resistance (contact pressure) at ULS can be designed for **200 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **5.0 MPa/m** for a contact pressure of **125 kPa**. The raft foundation design considers the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

A raft designed using the parameters provided above will be subjected to potential post- construction total and differential settlements of 25 and 20 mm, respectively.

Permissible Grade Raise Recommendations

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **1.0 m** is recommended. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise calculations.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

Lateral Support

The bearing medium under footing- and raft-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 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 of the same or higher capacity as the bearing medium soil.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** if referencing the Ontario Building Code (OBC) 2012, and **Class X_C** if referencing the OBC 2024. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

The native soil subgrade will provide a suitable founding medium for the basement floor slab. It is anticipated that the basement area for the proposed building will be mostly parking and the recommended pavement structures noted in Section 5.7 will be applicable.

However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

Any soft areas in the basement slab subgrade should be removed and backfilled with appropriate backfill material prior to placing fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, is recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

In consideration of the groundwater conditions at the site, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the subfloor fill under the lower basement floor. This is discussed further in Section 6.1.

5.6 Basement Wall

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

The total earth pressure (P_{AE}) includes the static earth pressure component (P_o) and the seismic component (ΔP_{AE}).

Lateral Earth Pressures

The static horizontal earth pressure (P_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the "at-rest" case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

The peak ground acceleration, (a_{max}), for the area of the subject site, is 0.353 g according to the OBC 2024. 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 pressures calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per the OBC 2024.

5.7 Pavement Design

Lowest Underground Parking Level

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

Table 2 - Recommended Rigid Pavement Structure - Lower Parking Level	
Thickness (mm)	**Material Description**
125	**Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

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

Pavement Structure Over Podium Deck

The pavement structures presented in Tables 3 and 4 should be used for car only parking areas, at grade access lanes and heavy loading parking areas over the top of the podium structure, should they be required.

Table 3 - Recommended Pavement Structure - Car Only Parking Areas Over Podium Deck	
Thickness (mm)	**Material Description**
50	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
200*	**BASE** - OPSS Granular A Crushed Stone
See below**	**Thermal Break** -** Rigid Insulation (See Following Paragraph)
n/a	**Waterproofing Membrane and IKO Protection Board**
SUBGRADE – Reinforced concrete podium deck * Thickness of base course is dependent on grade of insulation as noted in proceeding paragraph ** If specified by others, not required from a geotechnical perspective	

Table 4 - Recommended Pavement Structure – Access Lanes, Fire Truck Lane, Ramp, and Heavy Loading Areas Over Podium Deck	
Thickness (mm)	**Material Description**
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – HL-8 or Superpave 19.0 Asphaltic Concrete
300*	**BASE** - OPSS Granular A Crushed Stone
See below**	**Thermal Break** -** Rigid Insulation (See Following Paragraph)
n/a	**Waterproofing Membrane and IKO Protection Board**
SUBGRADE – Reinforced concrete podium deck * Thickness of base course is dependent on grade of insulation as noted in proceeding paragraph ** If specified by others, not required from a geotechnical perspective	

The transition between the pavement structure over the podium deck subgrade and soil subgrade beyond the footprint of the podium deck is recommended to be transitioned to match the pavement structures provided in the following section.

For this transition, a 5H:1V is recommended between the two subgrade surfaces. Further, the base layer thickness should be increased to a minimum thickness of 500 mm below the top of the podium slab a minimum of 1.5 m from the face of the foundation wall prior to providing the recommended taper.

Should the proposed podium deck be specified to be provided a thermal break by the use of a layer of rigid insulation below the pavement structure, its placement within the pavement structure is recommended to be as per the above-noted tables. The layer of rigid insulation is recommended to consist of a DOW Chemical High-Load 100 (HI-100), High-Load 60 (HI-60), or High-Load 40 (HI-40). The base layer thickness will be dependent on the grade of insulation considered for this project and should be reassessed by the geotechnical consultant once pertinent design details have been prepared.

The higher grades of insulation have more resistance to deformation under wheel-loading and require less granular cover to avoid being crushing by vehicular loading. It should be noted that SM (Styrofoam) rigid insulation is **not** considered suitable for this application.

Pavement Structure on Overburden Soils

The following pavement structures may be considered for at-grade car only parking and heavy traffic areas, should they be required. The proposed pavement structures are shown in Tables 5 and 6, on the next page.

Table 5 - Recommended Pavement Structure - Car-Only Parking Areas	
Thickness (mm)	**Material Description**
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soils, bedrock or OPSS Granular B Type I or II material placed over in situ soil or bedrock	

Table 6 - Recommended Pavement Structure - Heavy-Truck Traffic and Loading Areas	
Thickness (mm)	**Material Description**
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soils, bedrock or OPSS Granular B Type I or II material placed over in situ soil or bedrock	

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 and Backfill

Foundation Drainage

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

However, if the foundation walls will be poured in a blind-side fashion, the perimeter drainage pipe should be placed within the interior of the building below the basement floor slab and provided with a gravity connection to the building's storm sump pit. Further, drainage sleeves cast in the foundation wall at the top of footing level should be placed at approximate 3 m centres around the perimeter of the building, to connect the foundation drainage board to the interior perimeter drainage pipe.

Underfloor Drainage

Underfloor drainage will be required to control water infiltration. For design purposes, we recommend that 150 mm diameter perforated pipes be placed at 6 m centers below the lowest level floor slab. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Elevator (and Sump) Pit Waterproofing

All elevator shaft exterior foundation walls and floor slabs should be waterproofed to avoid any infiltration into the elevator pit.

It is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved equivalent), is applied to the exterior of the elevator shaft foundation wall. The membrane should extend to the top of the footing in accordance with the manufacturer's specifications.

A continuous PVC waterstop, such as Southern Waterstop 14RCB (or approved equivalent), should be installed within the interface between the concrete base slab below the elevator pit sidewalls. An outlet for any trapped water should be installed through the elevator pit wall with a gravity connection to the underfloor drainage system or directly to the sump pit.

A protection board should be placed over the waterproofing membrane to protect the membrane from damage during the backfilling operations. Consideration should also be given to waterproofing the sump pit(s). If chosen, the above-noted waterproofing methodology will also be applicable to sump pit waterproofing.

Foundation Backfill

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

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover alone, or a combination of soil cover in conjunction with foundation insulation, should be provided in this regard.

Other exterior unheated footings (such as those for isolated exterior piers, or underground parking garage access ramp footings) or footings adjacent to garage bay doors which may be open to exterior conditions for extended periods of time (such as the entrance to the underground parking garage) are more prone to deleterious movement associated with frost action than the exterior walls of the proper structure. These footings should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

6.3 Excavation Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

Temporary Side Slopes

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level.

The subsurface soil 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 maintain a safe working distance 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 excavation where insufficient room is available for open cut methods.

The shoring requirements designed by Paterson, or a structural engineer specializing in those works and approved by Paterson, will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team.

Inspections of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural designer prior to implementation.

The temporary shoring system could consist of a soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, neighboring buildings, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced.

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

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

Table 7 – 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 no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated full weight, with no hydrostatic groundwater pressure component.

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent material specifications and standard detail drawings from the department of public works and services, infrastructure services branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. 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 or Granular B Type II with a maximum size of 25 mm. The bedding layer should be increased to a minimum thickness of 300 mm where the subgrade consists of grey silty clay.

The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the SPMDD.

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

Clay Seals

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, sub-bedding and cover material.

The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Provisions should be carried out for using higher capacity open sump systems for excavations undertaken below the bedrock surface.

The contractor should be prepared to direct water away from all 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.

Impacts on Neighboring Properties

The excavation for the proposed building is not expected to extend significantly below the groundwater level. Therefore, impacts to adjacent properties are not expected as a result of minor, localized dewatering which may occur at this site.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

Precaution must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (GU – General Use cement) would be appropriate for this site. The chloride content and pH of the sample indicate that they are not a significant factor in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of aggressive to very aggressive corrosive environment.

6.8 Tree Planting Restrictions

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks applicable if the building is founded on a silty clay bearing surface.

The following tree planting setbacks are recommended for this site. Tree planting setback limits may be reduced to **7.5 m** for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m), as measured from the proposed building, provided the following criteria are met.

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

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

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

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

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

It should be noted that shrubs and other small plants with roots less than 1m deep are permitted within the 7.5 m setback area.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and/or detailed designs of the proposed development have been prepared:

- ❑ Review detailed grading, servicing, landscaping, and structural plan(s) from a geotechnical perspective.

- ❑ Review of the geotechnical aspects of the excavation contractor's shoring design, if not designed by Paterson, prior to construction, if applicable.

In addition, it is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

- ❑ Review and inspection of the installation of the foundation waterproofing and drainage systems.

- ❑ 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.

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

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

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Mr. Shiv Bhasker, 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.

Nicole Patey, P.Eng.

Scott. S. Dennis, P.Eng.

Report Distribution:

❑ Mr. Shiv Bhasker (Email Copy)
❑ Paterson Group (1 Copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1773 & 1767 Baseline Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 363262.86 **NORTHING:** 5024226.40 **ELEVATION:** 87.84

PROJECT: Proposed Residential Development

BORINGS BY: CME-55 Low Clearance Drill

REMARKS:

DATE: February 18, 2025

FILE NO. : PG7446

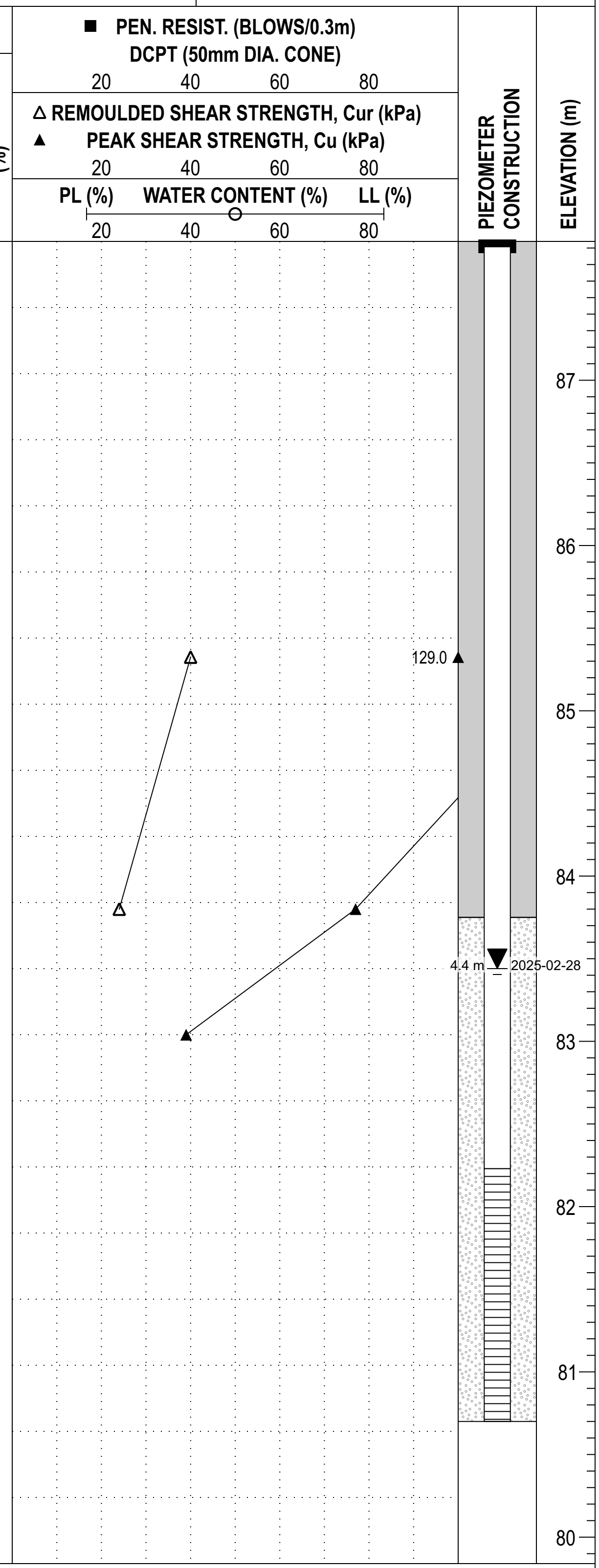
HOLE NO. : BH 1-25

SAMPLE DESCRIPTION (GROUND SURFACE)	Bottom depth [elevation]
ASPHALT	0.06m [87.78m]
FILL: Compact, brown silty clay, with sand and gravel, trace brick fragments	0.28m [87.56m]
FILL: Brown silty clay, trace gravel	2.21m [85.63m]
Very stiff to firm, brown **SILTY CLAY**, trace gravel and sand seams - Grey and silt content increased by 3.73 m depth	5.26m [82.58m]
GLACIAL TILL: Compact, grey silty clay, with gravel, cobbles and boulders	7.14m [80.70m]
End of Borehole Practical refusal to augering at 7.14 m depth (GWL at 4.40 m depth - February 28, 2025)	

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			
SS 2	33	3-7-5-5 12	
SS 3	25	2-2-3-2 5	
SS 4	25	P	
SS 5	8	3-2-2-2 4	
SS 6	33	P	
SS 7	0	P	
SS 8	50	P	
SS 9	42	2-5-7-8 12	
SS 10	34	11-50-/-/ 50/0	

STRATA PLOT

DEPTH (m)



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SOIL PROFILE AND TEST DATA

PATERSON GROUP

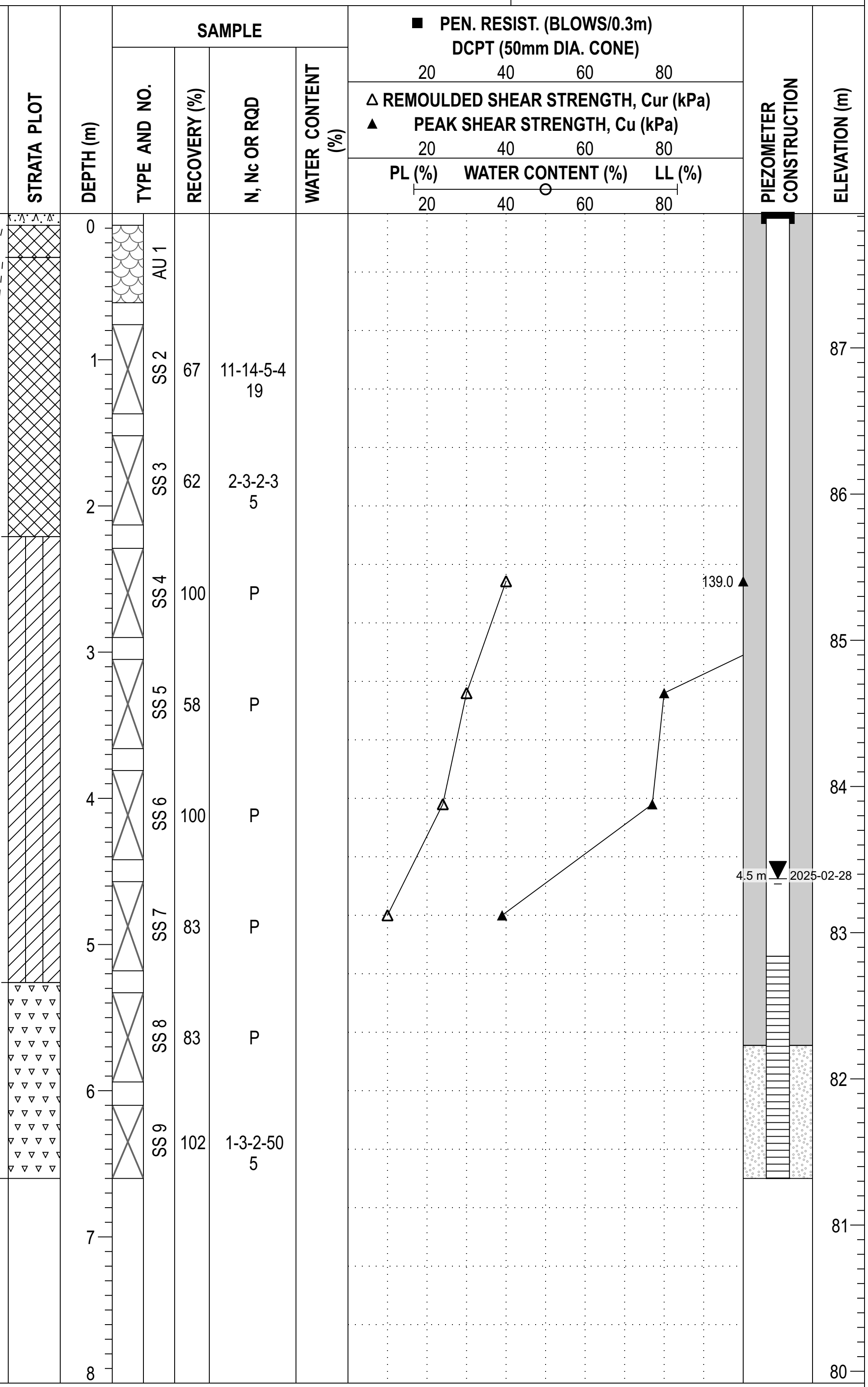
Geotechnical Investigation
1773 & 1767 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 363290.35 **NORTHING:** 5024238.91 **ELEVATION:** 87.92

PROJECT: Proposed Residential Development
BORINGS BY: CME-55 Low Clearance Drill
REMARKS:
DATE: February 18, 2025

FILE NO. : PG7446
HOLE NO. : BH 2-25

SAMPLE DESCRIPTION	Depth / Elevation	TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
GROUND SURFACE	0				
ASPHALT	0.08m [87.84m]	AU 1			
FILL: Compact, brown silty clay, with sand and gravel	0.30m [87.62m]				
FILL: Brown silty clay, trace gravel		SS 2	67	11-14-5-4 19	
	2.21m [85.71m]	SS 3	62	2-3-2-3 5	
Very stiff to firm, brown **SILTY CLAY**		SS 4	100	P	
		SS 5	58	P	
- Grey by 3.73 m depth		SS 6	100	P	
	5.26m [82.66m]	SS 7	83	P	
GLACIAL TILL: Compact, grey silty clay, with gravel, cobbles and boulders		SS 8	83	P	
	6.60m [81.32m]	SS 9	102	1-3-2-50 5	
End of Borehole					
Practical refusal to augering at 6.63 m depth					
(GWL at 4.55 m depth - February 28, 2025)					



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

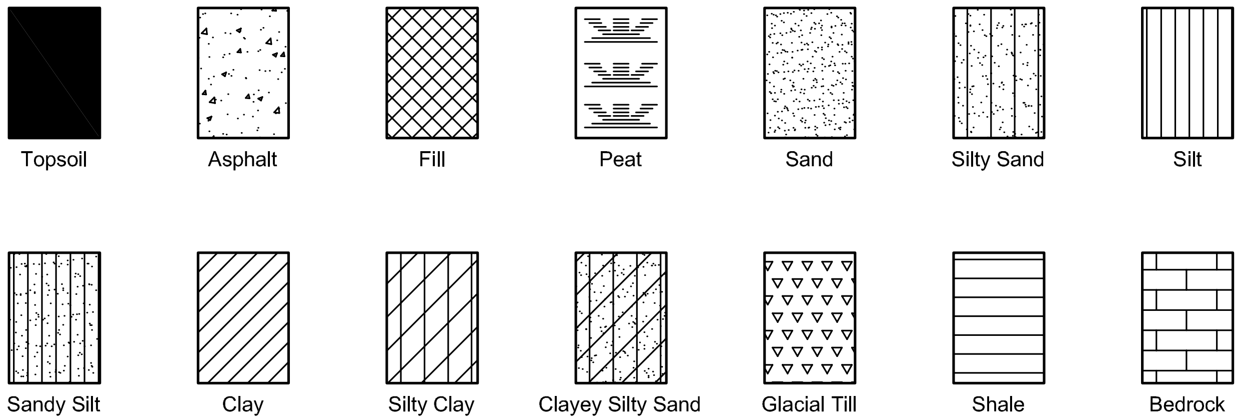
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

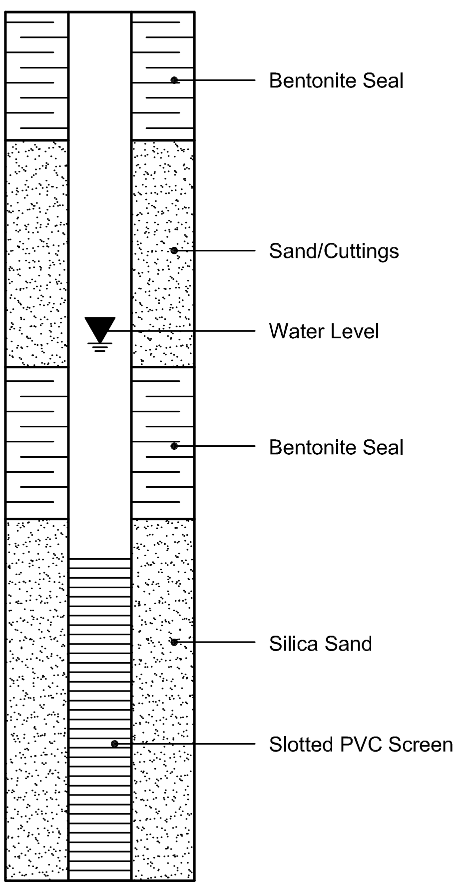
SYMBOLS AND TERMS (continued)

STRATA PLOT

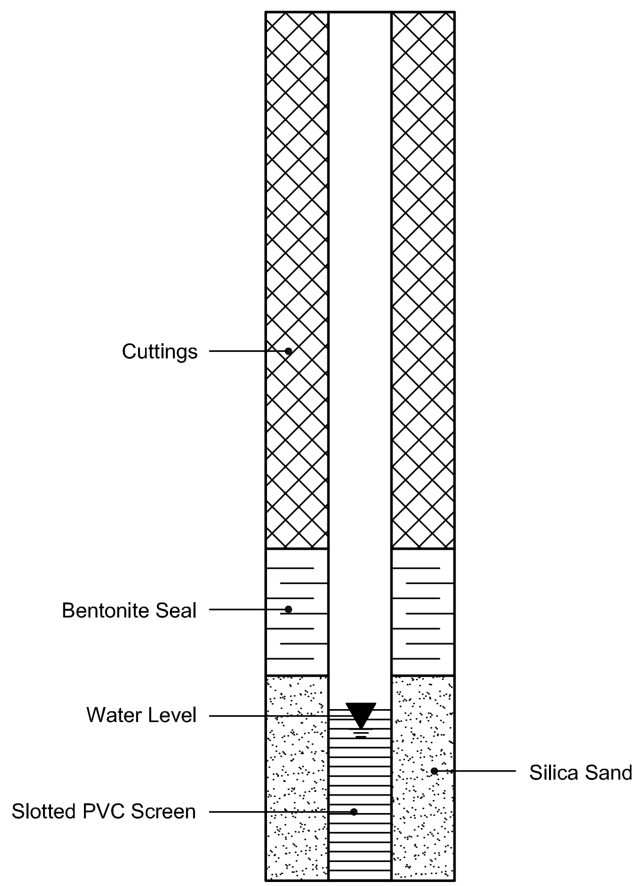


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



	Client ID:	BH1-25-SS4	-	-	-		
	Sample Date:	18-Feb-25 09:00	-	-	-	-	-
	Sample ID:	2508160-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	81.8	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.69	-	-	-	-	-
Resistivity	0.1 Ohm.m	10.7	-	-	-	-	-
Anions							
Chloride	10 ug/g	139	-	-	-	-	-
Sulphate	10 ug/g	987	-	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG7446-1 – TEST HOLE LOCATION PLAN

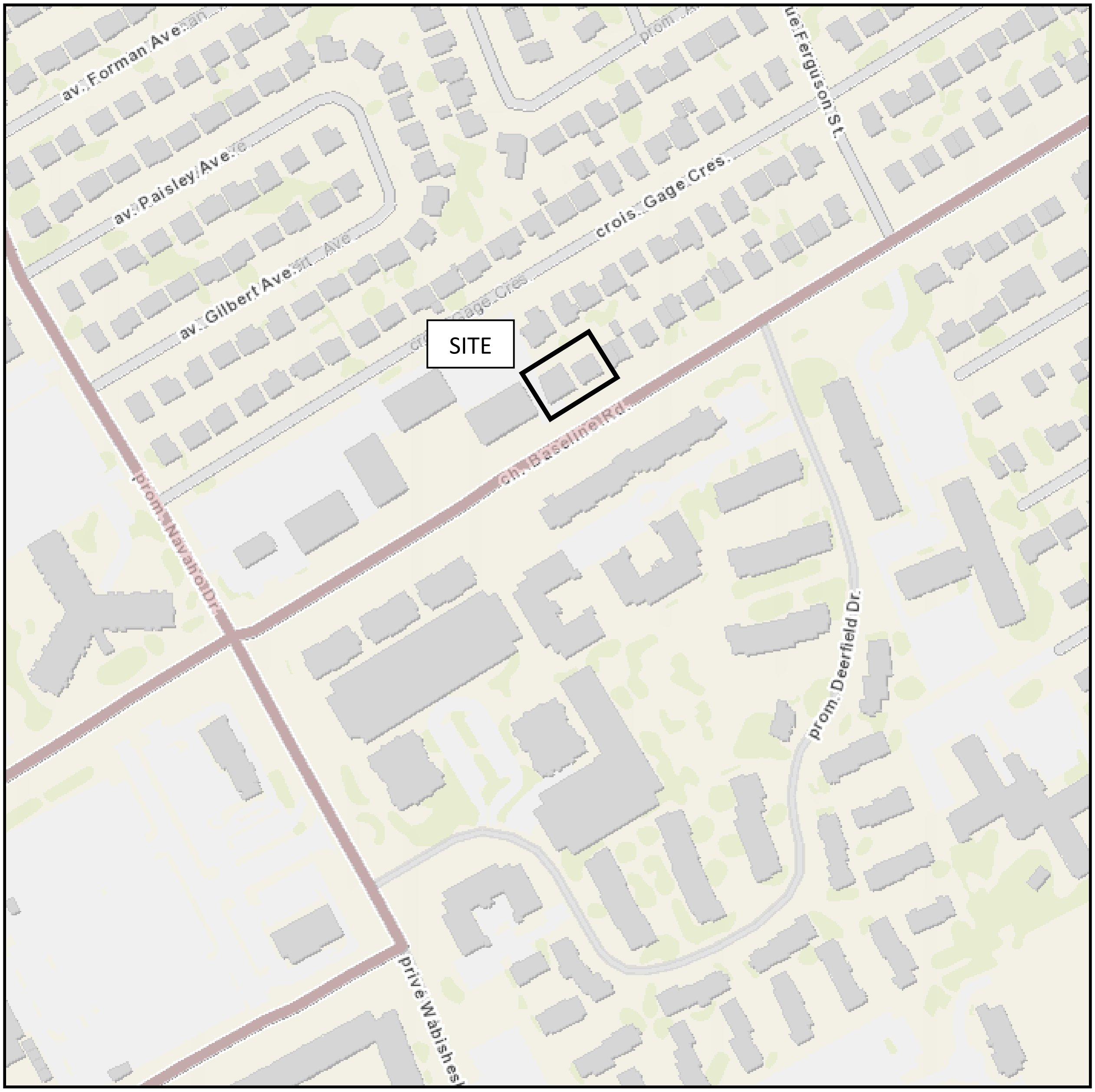
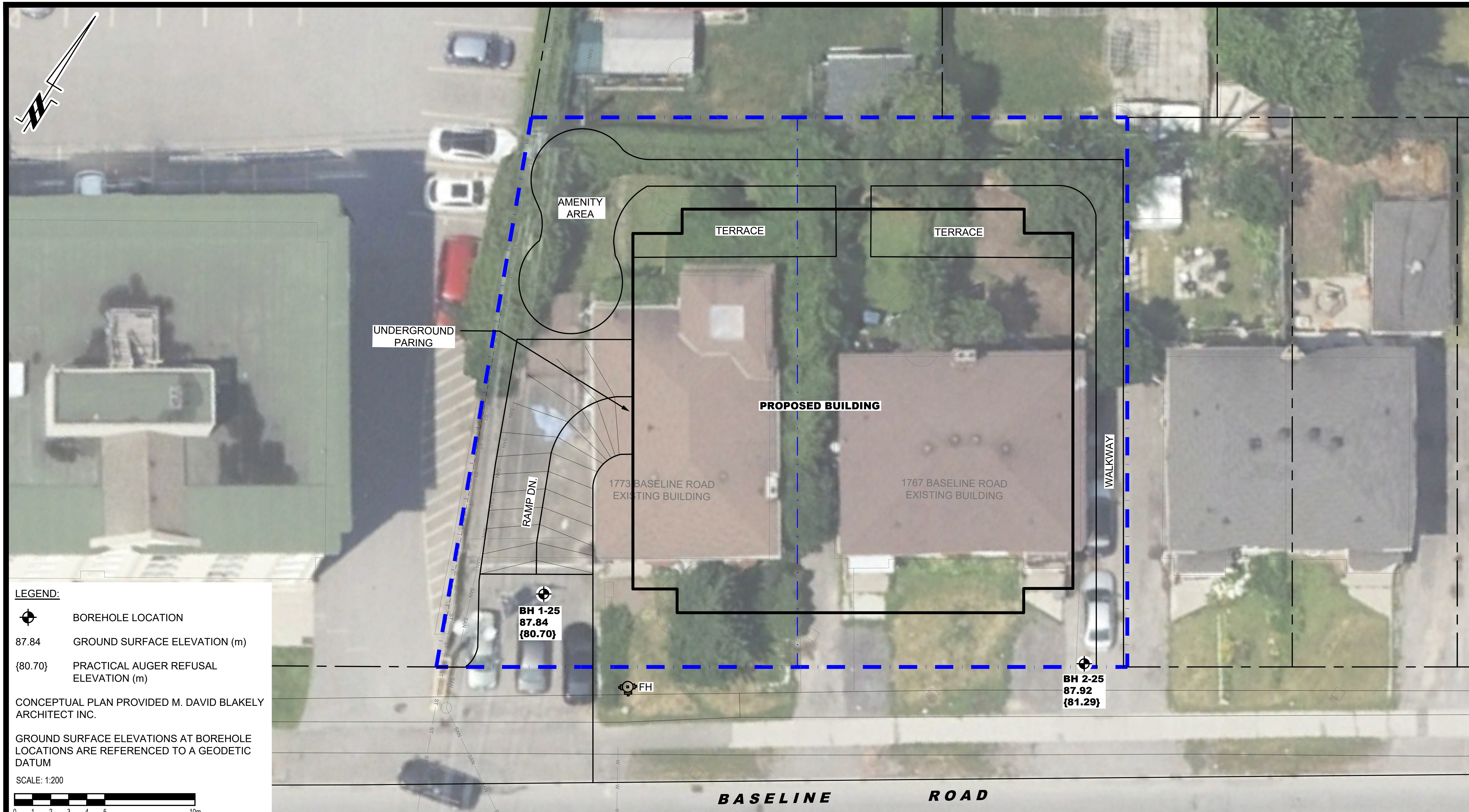


FIGURE 1

KEY PLAN

PATERSON GROUP



LEGEND:

⌖	BOREHOLE LOCATION
87.84	GROUND SURFACE ELEVATION (m)
{80.70}	PRACTICAL AUGER REFUSAL ELEVATION (m)

CONCEPTUAL PLAN PROVIDED M. DAVID BLAKELY ARCHITECT INC.

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

SCALE: 1:200

0 1 2 3 4 5 10m

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TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

MR. SHIV BASKER c/o DCR PHOENIX HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
1773 AND 1767 BASELINE ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:200	Date: 03/2025
Drawn by: YA	Report No.: PG7446-1
Checked by: NP	Dwg. No.: PG7446-1
Approved by: SD	Revision No.:

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11x17