

Geotechnical Investigation

Proposed Residential Development

466 Tremblay Road & 1440 St. Laurent Blvd.
Ottawa, Ontario

Prepared for Rohit Homes

Report PG7666-1 dated August 7, 2026



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1.0 Introduction

Paterson Group (Paterson) was commissioned by Rohit Homes to conduct a geotechnical investigation for the proposed residential development to be located at 466 Tremblay Road & 1440 St. Laurent Blvd. in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of test holes.
- ☐ 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 the presence or potential presence of contamination on the subject site was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the current conceptual drawings, it is our understanding that the proposed development will consist of multiple multi-storey residential buildings with one level of shared underground parking which will occupy the majority of the subject site. Local access lanes and landscaped areas are also proposed.

Demolition of the existing building will be required for development of the subject site. 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 investigation was carried out on September 5, 2025 and consisted of advancing a total of 4 boreholes to a maximum depth of 7.6 m below the existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground utilities and available access.

A previous geotechnical investigation, which consisted of advancing a total of 3 boreholes to a maximum depth of 5.5 m below existing grade, was completed at the subject site by others in December, 2021. The borehole locations are shown on Drawing PG7666-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In-Situ Testing

Soil samples were recovered from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using a 47.6 mm inside diameter coring equipment. All soil samples were visually inspected and classified on site. The auger and split spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split-spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples and 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.

Bedrock samples were recovered from boreholes BH 1-25, BH 2-25 and BH 3-25 using a core barrel and diamond drilling techniques. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. The values are indicative of the bedrock quality.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

A groundwater monitoring well was installed in borehole BH 2-25 during the geotechnical investigation to permit monitoring of groundwater levels subsequent to the completion of the sampling program. Flexible piezometers were installed in boreholes BH 1-25 and BH 3-25. The groundwater observations are discussed in Section 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The borehole locations, and ground surface elevation at each borehole location, were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The locations of the boreholes, and ground surface elevation at each borehole location, are presented on Drawing PG7666-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site 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 1 month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The northern portion of the subject site is occupied by a single-storey commercial building with associated asphalt paved access lanes, parking areas and landscaped areas. The southern portion of the site consist of an asphalt-paved parking lot associated with the existing development.

The property is bordered by Tremblay Road to the north, vacant, undeveloped lands to the east and south, and by residential buildings to the west. The ground surface across the subject site is generally flat and at grade with adjacent roadways at geodetic elevations of 68.0 to 68.5 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the subject site consists of asphaltic concrete underlain by an approximate 0.2 to 1.7 m thickness of fill material. The fill material was observed to consist of brown silty sand with crushed stone and/or brown silty clay with sand, gravel and crushed stone.

The fill material was generally underlain by a thin layer of compact, brown silty sand with trace to some clay at boreholes BH 3-25 and BH 4-25, extending to approximate depths of 1.7 to 1.5 m below the existing ground surface, respectively.

A layer of glacial till was encountered below the fill and/or silty sand deposit at boreholes BH 1-25 through BH 3-25, extending to approximate depths of 1.9 to 2.2 m below the existing ground surface. The glacial till was generally observed to consist of compact to very dense brown silty sand with clay, gravel, cobbles and boulders.

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

Bedrock

Weathered bedrock was encountered at approximate depths ranging from 1.5 to 2.2 m below the existing ground surface. At borehole BH1 and BH3 by others, the weathered bedrock was augered through to approximate depths of 2.8 and 2.9 m, respectively.

At borehole BH 4-25, practical refusal to augering was encountered at 1.5 m below the existing ground surface.

The bedrock was cored at boreholes BH 1-25, BH 2-25, BH 3-25 and BH 2 and based on the RQDs of the recovered bedrock core, was generally weathered and of very poor quality to approximate depths ranging from 3.2 to 4.7 m, becoming fair to excellent in quality with depth. The bedrock was cored to a maximum depth of 7.6 m

Based on available geological mapping, the subject site is located in an area where the bedrock consists of shale of the Carlsbad Formation.

4.3 Groundwater

Groundwater levels were measured in the installed monitoring well and piezometers on September 11, 2025. The observed groundwater levels are summarized in Table 1 below.

Table 1 - Summary of Groundwater Levels				
Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Levels		Date Recorded
		Depth (m)	Elevation (m)	
BH 1-25	68.22	2.27	65.95	September 11, 2025
BH 2-25*	68.24	2.38	65.86	
BH 3-25	68.24	2.31	65.93	
Notes: The ground surface elevation at each borehole location for the current investigation was surveyed using a handheld GPS referenced to a geodetic datum. ** – Denotes Monitoring				

Groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the groundwater table can generally be expected within the bedrock at approximate depths of **2 to 3 m** below the ground surface. The recorded groundwater levels are also provided on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

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

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed residential development. It is recommended that the proposed buildings be founded on conventional spread footings bearing on the undisturbed weathered bedrock, or on clean, surface-sounded bedrock.

Bedrock removal will be required to complete the underground parking level. Further, expansive shale bedrock could be present on site, and precautions should be provided during construction to reduce the risks associated with the potentially heaving shale bedrock, if exposed during construction.

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

5.2 Site Grading and Preparation

Stripping Depth

Asphalt, topsoil and fill, such as those containing significant organic or deleterious material, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Due to the relatively shallow depth to bedrock and the anticipated founding level for the proposed buildings, all existing overburden material will be excavated from within the proposed building footprints.

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

Bedrock removal can be accomplished by hoe ramming where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming where large quantities of bedrock need to be removed.

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 placed for grading beneath the building area 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 fill material should be tested and approved prior to delivery.

The fill should be placed in maximum 300 mm thick lifts and compacted by suitable compaction equipment. Fill placed beneath the buildings should be compacted to a minimum of 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level of areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of the SPMDD.

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

Expansive shale deteriorates upon exposure to air and is not generally suitable for reuse as an engineered fill. The use of imported granular fill is recommended for this purpose.

Lean Concrete Filled Trenches

Where rock overbreak occurs at the underside of footing (USF) elevation, or where weathered shale is encountered at the USF, lean concrete (minimum **17 MPa** 28-day compressive strength) can be used to raise the subgrade from the clean, surface-sounded bedrock to the USF elevation. 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.

5.3 Foundation Design

Bearing Resistance Values

Footings placed directly on the undisturbed, weathered bedrock, or on lean concrete, which is placed directly over the undisturbed, weathered bedrock, can be designed using a bearing resistance value at serviceability limit states (SLS) of **500 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **750 kPa**.

Footings placed on clean, surface-sounded shale bedrock, or on lean-concrete trenches which are placed on clean, surface-sounded shale bedrock can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,200 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance at ULS.

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

Footings supported on clean, surface-sounded bedrock, and designed for the bearing resistance values provided herein, will be subjected 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. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

A soil bearing medium, or a heavily fractured, weathered bedrock bearing medium, will require a lateral support zone of 1H:1V (or flatter).

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class X_c**. If a higher seismic site class is required for the proposed buildings which are founded on, or within 3 m of, the bedrock surface, a site-specific shear wave velocity test should be completed in accordance with the Ontario Building Code (OBC) 2024 to accurately determine the applicable seismic site classification for foundation design of the proposed building.

Based on Paterson's review of the in-situ soil characteristics, the soils underlying the subject site are not considered susceptible to liquefaction.

5.5 Basement Floor Slab

With the removal of all topsoil and deleterious fill within the footprints of the proposed buildings, the bedrock will be considered an acceptable subgrade upon which to commence backfilling for the basement slab construction.

It is anticipated that the underground level for the proposed buildings will primarily consist of parking areas and the rigid pavement structure recommendation provided in Section 5.8 will be applicable. However, if storage or other uses of the lower level are considered where a concrete floor slab will be constructed, the upper 300 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, are 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 98% of the SPMDD.

In consideration of the groundwater conditions at the site and the depth of excavation, a sub-slab drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided under the lowest level floor slab. 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 proposed building. However, in our opinion, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m³.

The applicable effective unit weight of the retained soil can be estimated as 13 kN/m³, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

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

Static 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/m³)

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 $0.375 \cdot a_c \cdot H^2/g$ where:

$$a_c = (1.45 - a_{max}/g) a_{max}$$

γ = unit weight of fill of the applicable retained soil (kN/m³)

H = height of the wall (m)

g = gravity, 9.81 m/s²

The peak ground acceleration, (a_{max}), for the subject site is 0.30g according to 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 $P_o = 0.5 K_o \cdot \gamma \cdot H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2024.

5.8 Pavement Design

Underground Parking Level

For design purposes, it is recommended that the rigid pavement structure for the underground parking level of the proposed buildings consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The following rigid pavement structure is recommended.

Table 2 – Recommended Rigid Pavement Structure – Lowest Underground Parking Level	
Thickness (mm)	Material Description
150	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 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 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Podium Deck Area

It is anticipated that the podium deck structure will be provided car only parking areas, access lanes, fire truck lanes and loading areas. Based on the concrete slab subgrade, the pavement structure indicated in the Tables 3 and 4 may be considered for design purposes:

Table 3 – Recommended Pavement Structure – Car-Only Parking Areas (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 Paragraph Below)
n/a	Waterproofing Membrane and Protection Board
SUBGRADE – Reinforced Concrete Podium Deck *If specified by others, not required from a geotechnical perspective **Thickness is dependent on grade of insulation as noted in proceeding paragraph	

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

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 (HI-40). The pavement structures 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 Over Overburden

Beyond the podium deck, the following pavement structures may be considered for car only parking and heavy traffic areas. The proposed pavement structures are shown in Tables 5 and 6 on the following 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 fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, bedrock or fill.	

Table 6 – Recommended Pavement Structure – Access Lanes and Heavy Loading Area	
Thickness (mm)	Material Description
40	Wear Course – HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course – HL-8 or Superpave 19 Asphaltic Concrete
150	BASE – OPSS Granular A Crushed Stone
400	SUBBASE – OPSS Granular B Type II
Subgrade – Either fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil, bedrock or 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 I or II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment, noting that excessive compaction can result in subgrade softening.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed building. The system should consist of a 150 mm diameter perforated and corrugated PVC 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 a positive outlet, such as a gravity connection to the storm sewer.

Underslab Drainage System

An underslab drainage system is recommended to control water infiltration below the underground parking level slab. For preliminary design purposes, it is recommended that 150 mm perforated pipes be placed at approximate 6 m centres underlying the underground parking level slab. The spacing of the underslab drainage system should be confirmed by the geotechnical consultant at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials, such as clean sand or OPSS Granular B Type I granular material.

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 of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

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

However, the footings are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

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

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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

Temporary Shoring

Depending on the depth of excavation of the buildings and the proximity of the proposed building to the property boundaries, temporary shoring may be required to support the overburden soils of the adjacent properties. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also 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 temporary shoring system or soils supported by the system. Any changes to the approved temporary shoring system design should be reported immediately to the owner's structural designer prior to implementation.

The temporary shoring system may consist of a soldier pile and lagging system which could be cantilevered, anchored or braced. However, if the migration of off-site contamination is a concern, then a secant pile wall may be required along certain portions of the excavation. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring system be adequately supported to resist toe failure by means of rock bolts or extending into the bedrock through pre-augered holes if a soldier pile and lagging system is used.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressure acting on the shoring system may be calculated using 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
Unit Weight , kN/m ³	21
Submerged Unit Weight , 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 table.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures.

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 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 98% of the SPMDD. It should generally be possible to re-use materials above the cover material if the operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the 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 225 mm thick loose lifts and compacted to a minimum of 98% of the material's SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

Based on our observations, it is anticipated that groundwater infiltration into the excavation should be low to moderate and 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

Under the current regulations enacted by the Ministry of Environment, Conservation and Parks (MECP), any dewatering in excess of 50,000 L/day requires a registration on the Environmental Activity and Sector Registry (EASR), so long as that dewatering is related to construction. If the dewatering is not related to construction, a Permit to Take Water obtained from the MECP will be required.

In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of three to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan, to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. Should a Permit to Take Water be required, a minimum of five to six months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

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 in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

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

6.8 Protection of Potentially Expansive Shale Bedrock

Upon being exposed to air and moisture, the 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 20°C, 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.

The presence of expansive shale may be encountered at the subject site. To reduce the long term deterioration of the shale, exposure of the bedrock bearing surface to oxygen should be kept as low as possible. The bedrock bearing surface within the proposed building footprint and under infrastructure, where exposed, should be protected from excessive dewatering and exposure to ambient air. A 50 to 75 mm thick concrete mud slab, consisting of minimum 17 MPa lean concrete, should be placed on the exposed bedrock bearing surface within a 48 hour period of being exposed. As noted above, the excavated sides of the exposed bedrock may also need to 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.

- ☐ Review detailed grading plan(s) from a geotechnical perspective.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ 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 soils should be handled as per ***Ontario Regulation 406/19: On-Site and Excess Soil Management.***

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 Rohit 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.



Kevin A. Pickard, P.Eng.

Report Distribution:

- ☐ Rohit Homes (1 email copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

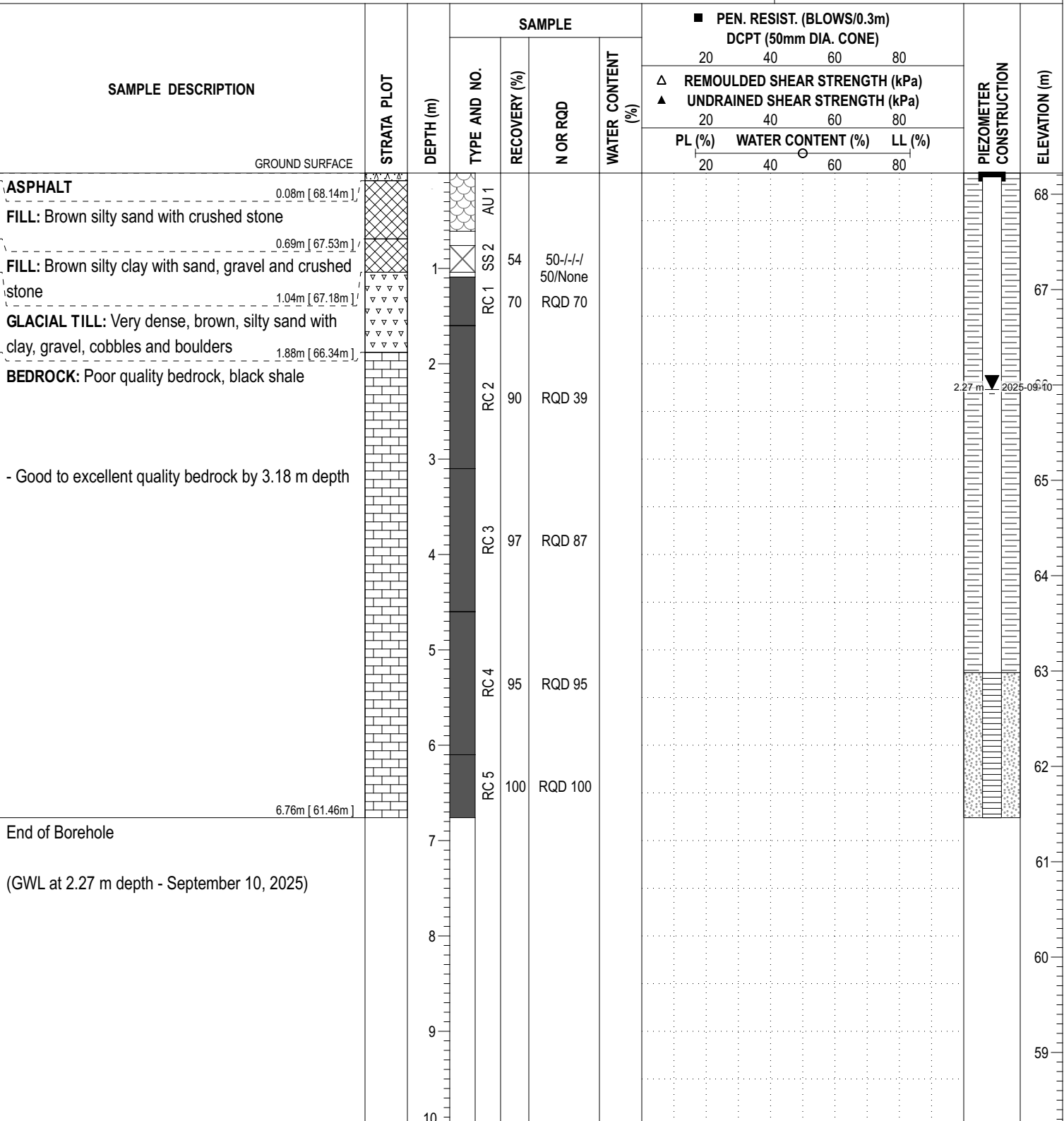
SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

LOG OF BOREHOLE SHEETS BY OTHERS

COORD. SYS.: MTM ZONE 9	EASTING: 372206.13	NORTHING: 5031195.55	ELEVATION: 68.22
--------------------------------	---------------------------	-----------------------------	-------------------------

PROJECT: Proposed Development ADVANCED BY: CME-55 Low Clearance Drill REMARKS:	FILE NO. : PG7666 HOLE NO. : BH 1-25
---	---

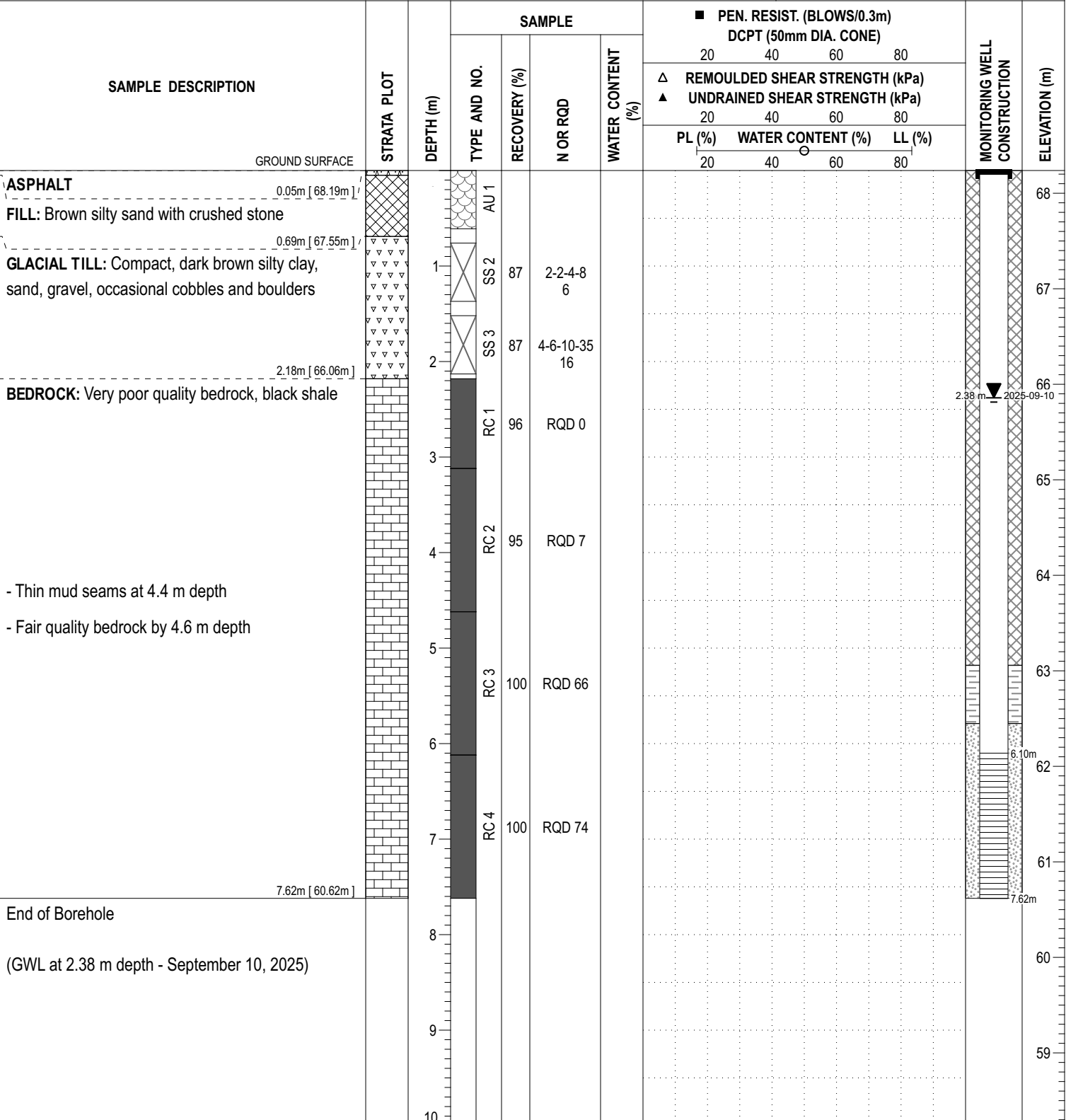
DATE: September 5, 2025


DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

COORD. SYS.: MTM ZONE 9 **EASTING:** 372202.27 **NORTHING:** 5031253.22 **ELEVATION:** 68.24

PROJECT: Proposed Development **FILE NO. :** PG7666

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS: **DATE:** September 5, 2025 **HOLE NO. :** BH 2-25


DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

ELEVATION: 68.24

HOLE NO. : BH 3-25

COORD. SYS.: MTM ZONE 9	EASTING: 372167.80	NORTHING: 5031331.54	ELEVATION: 68.26
--------------------------------	---------------------------	-----------------------------	-------------------------

PROJECT: Proposed Development ADVANCED BY: CME-55 Low Clearance Drill REMARKS:	FILE NO. : PG7666 HOLE NO. : BH 4-25
---	---

DATE: September 5, 2025

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE				■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)				PIEZOMETER CONSTRUCTION	ELEVATION (m)
			TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	20	40	60	80		
							△ REMOULDED SHEAR STRENGTH (kPa)					
							▲ UNDRAINED SHEAR STRENGTH (kPa)					
PL (%)	WATER CONTENT (%)		LL (%)									
20	40	60	80									
GROUND SURFACE												
ASPHALT			AU 1								68	
FILL: Brown silty sand with crushed stone												
0.08m [68.18m]												
0.69m [67.57m]												
Compact, brown SILTY SAND, some clay		1	SS 2	79	6-4-6-5 10						67	
1.52m [66.74m]												
End of Borehole												
Practical refusal to augering at 1.52 m depth on boulder or bedrock		2									66	
(BH blocked on - September 10, 2025)		3									65	
		4									64	
		5									63	
		6									62	
		7									61	
		8									60	
		9									59	
		10										

DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

BOREHOLE BH1

PROJECT: Proposed Residential Developments

CLIENT: Rohit Communities Ontario Inc.

LOCATION: 466 Tremblay Road

PENETRATION TEST HAMMER: 63.5 kg, Drop, 0.76 mm

PROJECT NUMBER: 211263

DATE OF BORING: 21-12-22

SHEET 1 of 1

DATUM:

DEPTH SCALE (meters)	SOIL PROFILE			SAMPLES			UNDIST SHEAR STRENGTH					DYNAMIC CONE PENETRATION TEST					MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION	
	DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	NUMBER	TYPE	BLOWS/0.3m	x Cu. kPa x											
								REM SHEAR STRENGTH											
								o Cu. kPa o					blows/300 mm						
		0	20	40	60	80	100	0	20	40	60	80	100						
	ASPHALTIC CONCRETE	0.00																	
	Grey crushed granular stone (FILL)	0.35																	
	TOPSOIL	0.50																	
1.0	Grey brown SILTY CLAY, trace to some sand and gravel	0.85			1	SS	11												
	Augered through Shale BEDROCK	1.52			2	SS	100												
2.0																			
					3	SS	100												
	End of borehole	2.82																	

Some groundwater observed at about 0.6 metres below existing ground surface at time of drilling, December 22, 2021.

DEPTH SCALE: 1 to 30

LOGGED: CI

BORING METHOD: Power Auger

AUGER TYPE: 200 mm Hollow Stem

CHECKED: SD

BOREHOLE BH2

PROJECT: Proposed Residential Developments

CLIENT: Rohit Communities Ontario Inc.

LOCATION: 466 Tremblay Road

PENETRATION TEST HAMMER: 63.5 kg, Drop, 0.76 mm

PROJECT NUMBER: 211263

DATE OF BORING: 21-12-22

SHEET 1 of 1

DATUM:

DEPTH SCALE (meters)	SOIL PROFILE			SAMPLES			UNDIST SHEAR STRENGTH					DYNAMIC CONE PENETRATION TEST					MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION	
	DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	NUMBER	TYPE	BLOWS/0.3m	x Cu. kPa x											
								REM SHEAR STRENGTH											
								o	Cu. kPa	o	blows/300 mm								
		0	20	40	60	80	100	0	20	40	60	80	100						
	ASPHALTIC CONCRETE	0.00																	
	Grey crushed granular stone (FILL)	0.30																	
1.0	Grey brown SILTY CLAY, trace to some sand and gravel	0.60			1	SS	6												
2.0	Cored through Shale BEDROCK	1.52			2	SS	100												
3.0																			
4.0																			
5.0																			
	End of borehole	5.51																	

Borehole dry to bedrock at time of drilling, December 22, 2021. Water level below bedrock surface could not be determined due to use of water in coring process.

DEPTH SCALE: 1 to 30

LOGGED: CI

BORING METHOD: Power Auger and Coring

AUGER TYPE: 200 mm Hollow Stem

CHECKED: SD

BOREHOLE BH3

PROJECT: Proposed Residential Developments
CLIENT: Rohit Communities Ontario Inc.
LOCATION: 466 Tremblay Road
PENETRATION TEST HAMMER: 63.5 kg, Drop, 0.76 mm

PROJECT NUMBER: 211263
DATE OF BORING: 21-12-22
SHEET 1 of 1
DATUM:

DEPTH SCALE (meters)	SOIL PROFILE			SAMPLES			UNDIST SHEAR STRENGTH						DYNAMIC CONE PENETRATION TEST						MOISTURE CONTENT (%)	PIEZOMETER OR STANDPIPE INSTALLATION	
	DESCRIPTION	DEPTH (m)	STRATA PLOT	ELEV. (m)	NUMBER	TYPE	BLOWS/0.3m	x Cu. kPa x													
								REM SHEAR STRENGTH													
								o Cu. kPa o						blows/300 mm							
							0	20	40	60	80	100	0	20	40	60	80	100			
	ASPHALTIC CONCRETE	0.00																			
	Grey crushed granular stone (FILL)	0.30																			
	Yellow brown to grey brown silty sand, trace to some clay and gravel (FILL)	0.55																			
1.0					1	SS	10														
					2	SS	7														
2.0																					
	Augered through Shale BEDROCK	2.03																			
					3	SS	100														
	End of borehole	2.90																			

Some  groundwater observed at a depth of about 1.5 metres below existing ground surface at time of drilling, December 22, 2021.

DEPTH SCALE: 1 to 30

LOGGED: CI

BORING METHOD: Power Auger

AUGER TYPE: 200 mm Hollow Stem

CHECKED: SD

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 = $(D_{30})^2 / (D_{10} \times D_{60})$
Cu	-	Uniformity coefficient = D_{60} / D_{10}

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

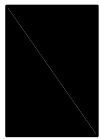
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void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

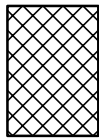
STRATA PLOT



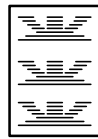
Topsoil



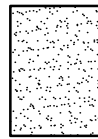
Asphalt



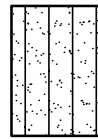
Fill



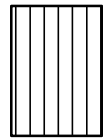
Peat



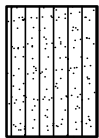
Sand



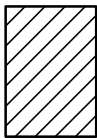
Silty Sand



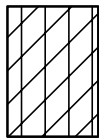
Silt



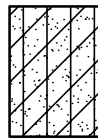
Sandy Silt



Clay



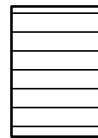
Silty Clay



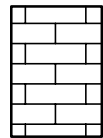
Clayey Silty Sand



Glacial Till



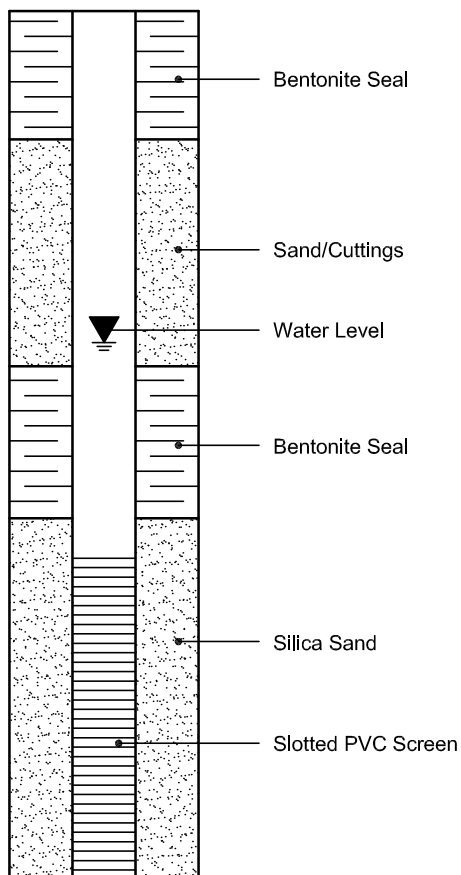
Shale



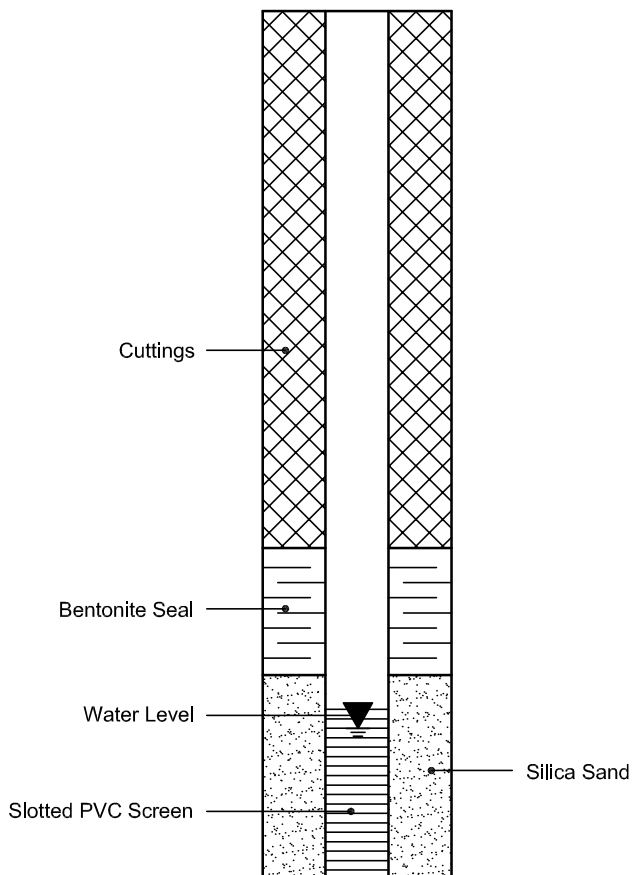
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 15-Sep-2025

Client: Paterson Group Consulting Engineers (Ottawa)

Order Date: 9-Sep-2025

Client PO: 63994

Project Description: PG7666

Client ID:	BH2-25-SS3 Top-5'-7'	-	-	-	-
Sample Date:	05-Sep-25 00:00	-	-	-	-
Sample ID:	2537213-01	-	-	-	-
Matrix:	Soil	-	-	-	-
MDL/Units					

Physical Characteristics

% Solids	0.1 % by Wt.	91.6	-	-	-	-
----------	--------------	------	---	---	---	---

General Inorganics

pH	0.05 pH Units	7.76	-	-	-	-
Resistivity	0.1 Ohm.m	29.7	-	-	-	-

Anions

Chloride	10 ug/g	22	-	-	-	-
Sulphate	10 ug/g	183	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG7666-1 – TEST HOLE LOCATION PLAN

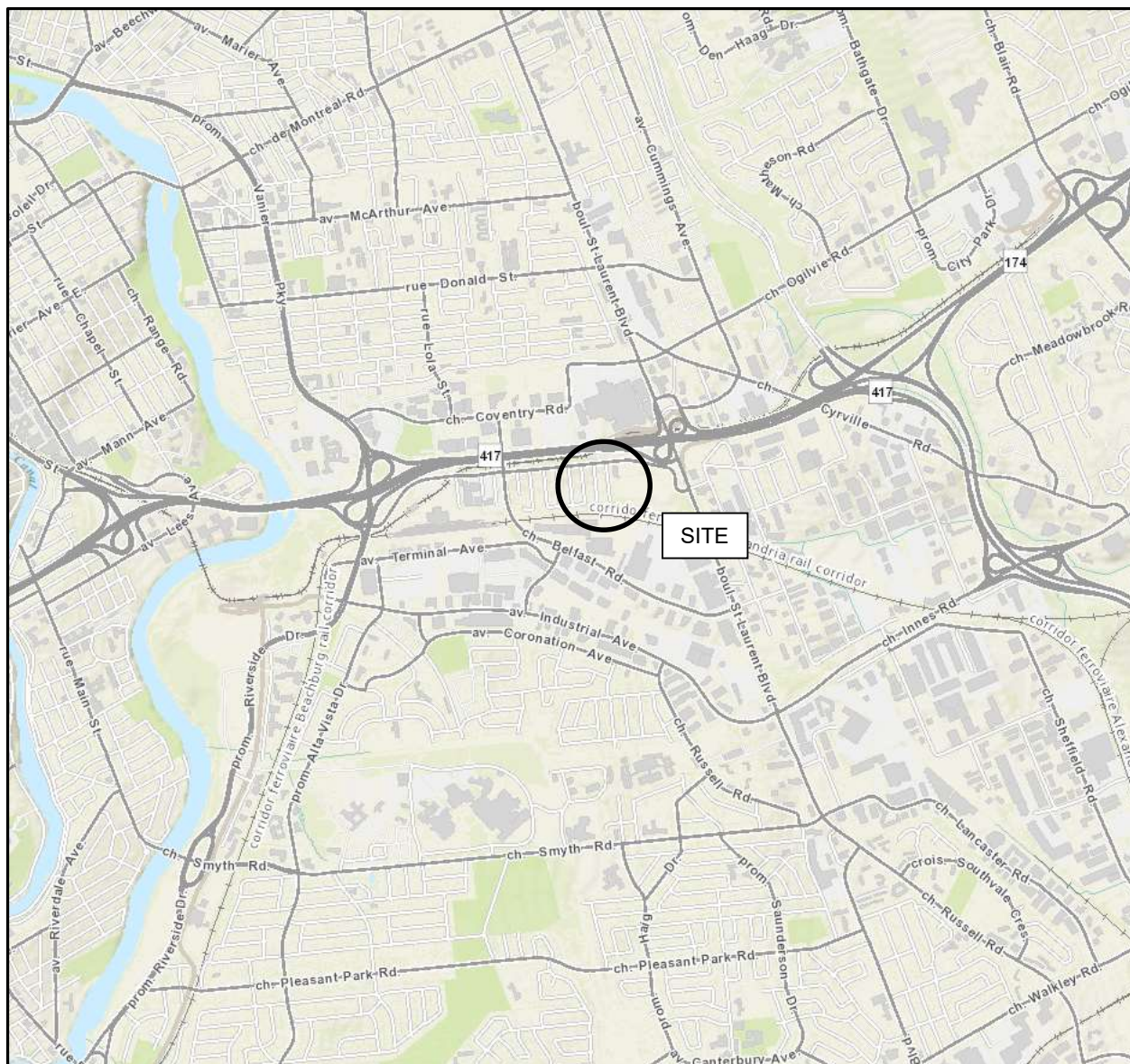
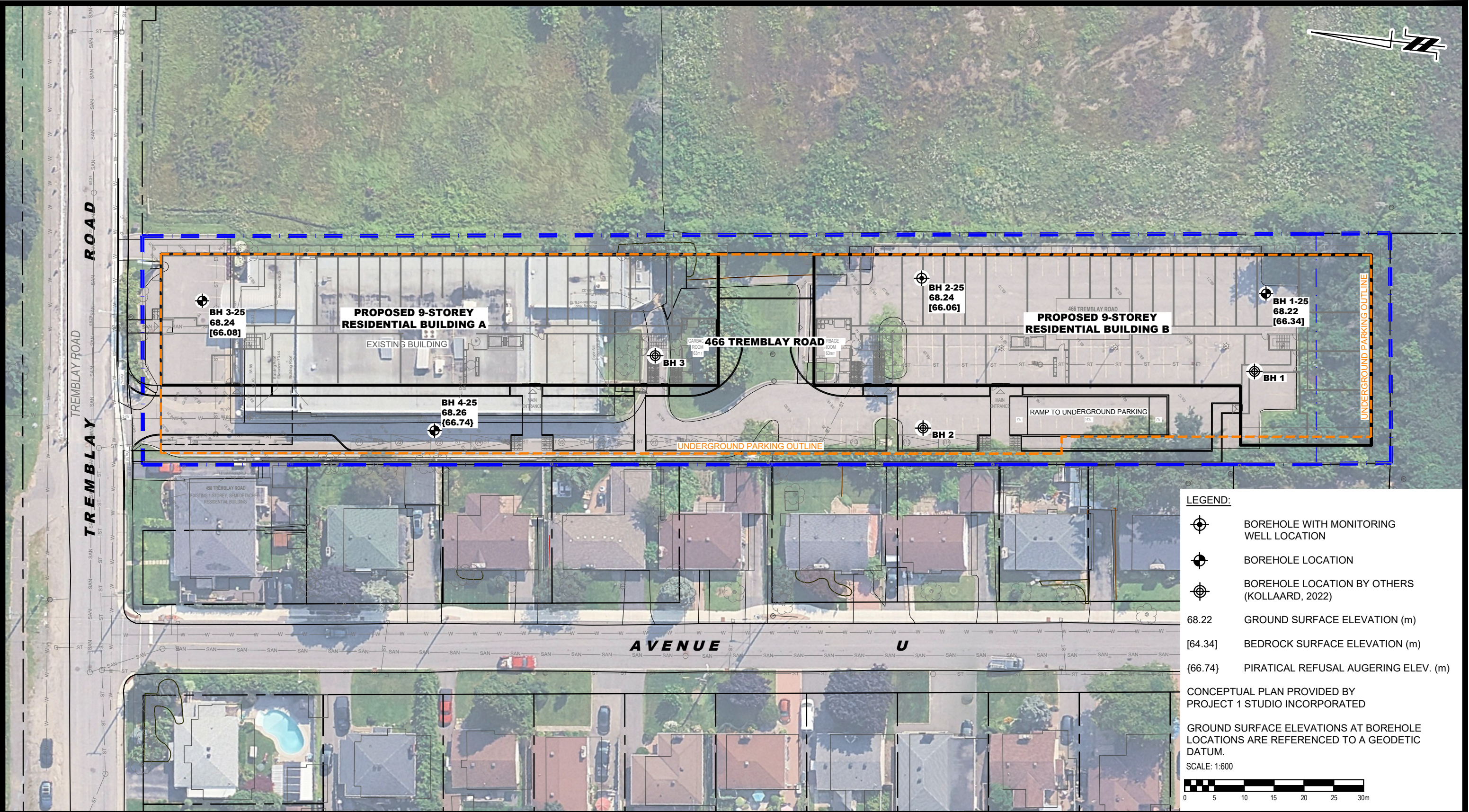


FIGURE 1

KEY PLAN



- LEGEND:**
- BOREHOLE WITH MONITORING WELL LOCATION
 - BOREHOLE LOCATION
 - BOREHOLE LOCATION BY OTHERS (KOLLAARD, 2022)
 - 68.22 GROUND SURFACE ELEVATION (m)
 - [64.34] BEDROCK SURFACE ELEVATION (m)
 - {66.74} PIRATICAL REFUSAL AUGERING ELEV. (m)

CONCEPTUAL PLAN PROVIDED BY
PROJECT 1 STUDIO INCORPORATED

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

SCALE: 1:600



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

1	UPDATED CONCEPTUAL PLAN	06/08/2026	MA
NO.	REVISIONS	DD/MM/YYYY	INITIAL

ROHIT COMMUNITIES INC.
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
466 TREMBLAY ROAD & 1440 ST. LAURENT BOULEVARD

OTTAWA,
Title:

ONTARIO

TEST HOLE LOCATION PLAN

Scale:	1:600	Date:	09/2025
Drawn by:	GK	Report No.:	PG7666-1
Checked by:	MA	Dwg. No.:	PG7666-1
Approved by:	KP	Revision No.:	1