Geotechnical Investigation

Proposed Multi-Storey Building

1299 Richmond Road
Ottawa, Ontario

Prepared for Brigil Construction

Report PG6598 – 1 Revision 1 dated September 6, 2024

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Brigil Construction to carry out a geotechnical investigation for the proposed multi-storey residential buildings to be located at 1299 Richmond Road, in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the current this geotechnical investigation was to:

- ❑ Determine the subsoil and groundwater conditions at this site by means of test holes.
- ❑ Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation.

2.0 Proposed Development

Based on available information, the proposed development will consist of two multi-storey residential and mixed-use buildings, with three underground levels. It is understood that the two buildings will be connected by a mid rise podium and will share a common substructure. The development will also include associated asphaltic parking areas, access lanes and landscaped areas. It is further anticipated that the site will be fully municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out between March 13 to March 15, 2023 and consisted of a total of five (5) boreholes, of which two (2) boreholes were advance to a maximum depths of 13.2 m, below the existing grade, where practical refusal to auguring was encountered; and three (3) of the boreholes were advanced to a maximum depths of 16.5 m, 15.1 m and 15.0 m, below the existing grade, cored and sampled approximately 2.0 to 3.0 m into the bedrock.

The boreholes were put down using a low clearance track-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer. The testing procedure for boreholes consisted of augering to the required depths and at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Borehole samples were recovered from a 50 mm diameter split-spoon (SS) or the auger flights (AU). All soil samples were visually inspected and initially classified on site. The split-spoon and auger samples were placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the split-spoon and auger samples were recovered from the test holes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

Rock samples were recovered from boreholes BH 1-23 and BH 3-23 using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes, and transported to Paterson's laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

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

Groundwater

Groundwater monitoring wells were installed in all the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson with respect to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG6598-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were collected from the subject site during the investigation and were visually examined in our laboratory to review the results of the field logging. Moisture content testing was performed on all the recovered field samples. The test results are included in Appendix 1.

All samples will be stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless 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 sample. If available, the results are presented in Appendix 1 and are discussed further in Subsection 6.8.

4.0 Observations

4.1 Surface Conditions

The site is currently occupied by a one-storey slab on grade commercial building. The site is separated from the residential developments to the west, high rise building to the north and west by Starflower Lane. The commercial development to the southwest is separated by Assaly Road and the commercial development to the south is separated by Richmond Road.

The site is almost fully covered by the building and associated asphalt parking lot. Richmond road is slightly elevated from the parking area while the surrounding roads slopes down to match the current elevation on site.

4.2 Subsurface Profile

Overburden

Fill

Generally, the subsurface profile was comprised of 50 to 130 mm asphaltic concrete, underlain by crushed stone with silty sand fill up to depths varying from 0.4 to 1 m at BH 1-23 to BH 4-23. Brown silty clay with some sand and gravel was observed underlying the crushed stone at BH 2-23 and BH 4-23 up to 1.6 m below the existing ground surface.

At BH 5-23, the subsurface profile comprised of 100 mm of topsoil, underlain by brown sandy silt with some clay, traces of gravel and organics.

Sandy Silt

Loose brown sandy silt with trace to some clay was generally observed underlying the fill at all the boreholes. The depth of the sandy silt layer varied from 2.2 to 3.1 m, with increasing clay content advancing deeper.

Silty Clay

A stiff to firm brown silty clay layer was encountered under the fill and silty. Seams of silty sand and layers with some sand seams were encountered. The brown silty clay was generally observed to be very stiff to stiff in consistency. The brown silty clay transitioned into a grey silty clay of stiff to firm consistency at depths of 4.7 to 5.5 m at BHs 2-23, 4-23 & 5-23.

Glacial Till

A compact to loose grey glacial till composed of sand in gravel in a silty clay soil matrix was found underlaying the silty clay deposit at depth of 9.3 to 12.0 m. The layer was noted to be highly saturated.

The glacial till became dense to very dense with depth at depths of 12.5 to 14.0 m. The silty clay matrix was noted to change to a silty sand matrix including gravel and cobbles. The layer was noted to be water bearing and highly permeable on site. No further testing was conducted to evaluate the hydraulic conductivity of the layer.

Bedrock

Based on the recovered core samples the bedrock was generally comprised of excellent quality grey quartz sandstone was encountered at the boreholes. Based on available geological mapping, the subject site is located in an area where the bedrock consists of the Ottawa Formation. The bedrock layer is expected to vary in depth from 12.0 m to 14.0 m below the existing grade.

4.3 Groundwater

Groundwater level readings were recorded on March 23, 2023, and are presented in Table 1 and on the Soil Profile and Test Data sheets in Appendix 1. It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. Additionally, groundwater levels are subject to seasonal fluctuations, therefore the groundwater levels could vary at the time of construction.

Long-term groundwater level can be estimated based on the observed color, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater is between 4.5 to 6.0 m.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH1-23	69.32	3.79	65.53	March 23, 2023
BH2-23	69.06	5.41	63.65	March 23, 2023
BH3-23	69.30	5.84	63.46	March 23, 2023
BH4-23	69.18	5.63	63.55	March 23, 2023
BH5-23	69.55	3.08	66.47	March 23, 2023

Note:
- The ground surface elevations are referenced to a geodetic datum.
- * Borehole with groundwater monitoring well

5.0 Discussion

5.1 Geotechnical Assessment

Foundation Design Considerations

From a geotechnical perspective, the subject site is suitable for the proposed multi-storey buildings. It is expected that the proposed building will be founded on a dense glacial till comprised of grey silty sand, some gravel, cobbles and boulders or on a limestone bedrock.

Alternately, to avoid excavating the entire building footprint to the bedrock level, footings could be placed over lean concrete infilled trenches. Near vertical, zero entry trench extending at least 300 mm beyond the footing face should be excavated to a clean bedrock surface approved by the geotechnical consultant. The trenches should be infilled by a minimum of 15 MPa lean concrete to the underside of the footing.

The above and other considerations are discussed in the following paragraphs.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant organic materials, should be stripped from under any buildings and other settlement sensitive structures. The existing fill material, where free of organic materials, should be reviewed by the geotechnical consultant at the time of construction to determine if the existing fill can be left in place below paved areas and below the slab granular fill layers.

Fill Placement

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 to the site. The fill should be placed in maximum 300 mm thick lifts and compacted to 98% of the material's standard Proctor maximum dry density (SPMDD).

Site-excavated soil can be placed as general landscaping fill where settlement is a minor concern of the ground surface. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm thick lifts and to a minimum density of 95% of the respective SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls due to the frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

The fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the paved areas should be compacted to at least 95% of its SPMDD.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming. However, no significant bedrock removal is expected due to the anticipated depth of the footings.

Vibration Considerations

Construction operations are 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 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. Therefore, all vibrations are recommended to be limited.

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

Bearing Surface Preparation

The excavation is expected to be completed below the groundwater table. Where the bearing surface will consist of glacial till, measures to protect against heaving and ground disturbance should be put in place. Accordingly, it is recommended that the entirety of each building footprint be excavated to the underside of footing elevation, and then covered with a 150 mm thick mud slab to protect the glacial till from disturbance.

Furthermore, groundwater pumping using dry wells with sump pumps which are located centrally within the excavation will be required to control the influx of water during construction. Details can be provided once the groundwater influx is better assessed during the excavation process.

Lean Concrete In-Filled Trenches

Where footings are designed to be supported on bedrock, and the bedrock is not encountered at the design underside of footing elevation, consideration should be given to excavating zero-entry vertical trenches to expose the underlying bedrock surface and then backfilling with lean concrete (15 MPa 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. The trench excavation should be at least 300 mm wider than all sides of the footing at the base of the excavation.

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 bedrock. The excavation bottom should be relatively clean using the hydraulic shovel only (no worker entry). Once approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

Footings placed on lean concrete filled trenches extending to the bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,000 kPa**. This is discussed further below.

5.3 Foundation Design

Foundation Option 1: Conventional Footings

Footings placed on an undisturbed, dense glacial till of silty sand matrix bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **400 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **700 kPa**. The provided bearing assumes a minimum depth of 10 m below existing grade. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS. Footings designed using the above-noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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 prior to the placement of concrete for footings.

Footings placed on the upper levels of the fractured bedrock a clean, surface sounded sandstone bedrock bearing surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,000 kPa**, incorporating a geotechnical resistance factor of 0.5. Alternately, footings can be placed over concrete in-filled (minimum 15 MPa) zero entry, near vertical trenches extended to a surface sounded bedrock bearing surface using the same bearing resistance values.

A factored bearing resistance value at ULS of **7,000 kPa,** incorporating a geotechnical resistance factor of 0.5, can be used for footings founded on clean, surface sounded bedrock and the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footing footprint(s). One drill hole should be completed per footing. The drill hole inspection should be completed by the geotechnical consultant.

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

Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Foundation Option 2: Raft Foundation

If the bearing resistance values are not sufficient for shallow foundation, raft foundation can be considered. The following parameters may be used for raft design and will apply for an undisturbed soil bearing surface. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings and approved by the geotechnical consultant.

Based on the following assumptions for a raft foundation, the proposed building can be designed with total and differential settlements of 25 and 15 mm, respectively.

For design purposes, it was assumed that the base of a raft foundation for a multi-storey building would be located at a depth of 9 to 11 m below existing ground surface and founded on glacial till for three (3) underground parking levels.

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 **500 kPa** will be considered acceptable.

The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The contact pressure provided considers the stress relief associated with the soil removal required for the proposed building.

The factored bearing resistance value at ULS can be taken as **750 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 **20 MPa/m** for a contact pressure of **750 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Foundation Option 3: End Bearing Piled Foundation

It is anticipated that the structure might require to be constructed over concrete filled steel pipe piles driven to refusal on the bedrock surface where the depth of the bedrock is located well below the proposed underside of footing for the development.

The bedrock surface is estimated to be located at a depth ranging from 8.2 to 16.7 m in depth throughout the site while the foundation for the development is anticipated at a depth of 11 to 14 m below the existing ground surface. The piles will need to be driven through a dense layer of glacial till below 11 to 14 m below existing ground surface.

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of two (2) to four (4) piles would be recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	1,000	1,250	6	27
245	11	1,150	1,450	6	31
245	13	1,300	1,600	6	35

The minimum recommended centre-to-centre pile spacing is 3 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.

Due to the presence of boulder pile driving could present as challenging, the installation method is the responsibility of the contractor to ensure all specification of the project are met.

Foundation Option 4: Drilled Shafts and Caissons

End bearing cast-in-place caissons can be used where supplemental axial resistance is required for structural design for the proposed building. The caisson should be installed by driving a temporary steel casing and excavating the soil through the casing. A minimum of 35 MPa concrete should be used to in fill the caissons. The caissons are to be structurally reinforced over their entire length.

Two conditions for drilled shafts are applicable for this site. The first alternative is a caisson installed on the sound bedrock augering through the weathered bedrock (end bearing).

The compressive resistance for such piles is directly related to the compressive strength of the bedrock. It is recommended that the entire capacity be derived from the end bearing capacity.

The second alternative is a concrete caisson socketed into bedrock. The axial capacity is increased by the shear capacity of the concrete/rock interface. Furthermore, the tensile resistance of the caisson is increased by the rock capacity. It should be noted that the rock socket should be reinforced.

Table 3 below presents the estimated capacity for different typical caisson sizes for a rock bearing caisson and rock socketed caisson extending 3 m into sound bedrock.

Table 3 - Caisson Pile Capacities

Caisson Diameter		Axial Capacity (kN)	
inch	mm	End Rock Bearing	Rock Socket
36	900	8,000	11,000
42	1,000	10,000	13,000
48	1,200	14,500	17,500
54	1,375	18,500	21,500
60	1,500	22,500	25,500

Notes:
- 3 m rock socket in bedrock
- Reinforced caisson and rock socket when applicable
- 0.4 geotechnical factor applied to the shaft capacity

The minimum recommended centre-to-centre pile spacing is 3 times the pile diameter. The closer the piles are spaced, however, the more potential that the installation of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously installed piles.

Option 5: Diaphragm/ Secant Wall Foundation

The multi-storey building will be founded on reinforced diaphragm walls or secant piles that are socketed within the bedrock along the perimeter of the parking garage. Furthermore, the interior portion of the building (shear walls, elevator shafts, stairwells and other portions of the structure selected by the structural engineer) will be founded on steel reinforced barrette walls socketed in the bedrock.

Table 4 – Bearing Resistance Values for Diaphragm Walls Socketed in Bedrock

Socketed Length (m)	Width of Diaphragm Wall (mm)	Bearing Resistance Value at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
1	600	2,000	2,500
	750	2,000	2,500
2	600	2,500	3,250
	750	2,500	3,250
3	600	3,000	3,500
	750	3,000	3,500

The diaphragm wall can also be designed using the bedrock shear strength if drilled and socketed within the bedrock. This shear strength value will be **500** to **800 kPa** and is reduced due to the fractured nature of the upper levels of the grey limestone layer.

The waterproofing and drainage approach should be adjusted with the method used as secant piles and slurry walls can leak slightly following installation.

Settlement

The total and differential settlement will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 to 20 mm, respectively.

A building founded on deep foundations or shallow foundations bearing on an acceptable bedrock bearing surface and designed for the bearing resistance value provided herein will be subjected to negligible potential postconstruction total and differential settlements.

Lateral Support

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

Above the groundwater level, adequate lateral support is provided to the in-situ bearing medium soils when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil.

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

A heavily fractured, weathered bedrock and/or overburden 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 C for the foundations anticipated at the subject site. The soils underlying the subject site are not susceptible to liquefaction.

A higher seismic site class, such as Class A or B, is available for design provided footings are extended within 3 from the bedrock surface and a site-specific seismic shear wave velocity test is conducted by the geotechnical consultant.

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

5.5 Basement Slab

The basement areas for the proposed project will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be constructed, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. The upper 200 mm sub-slab fill is recommended to consist of OPSS Granular A crushed stone for slab on grade construction.

All backfill material within the footprint of the proposed building(s) should be placed in a maximum of 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any 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(s) should be placed in a maximum of 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

In consideration of the groundwater conditions encountered at the time of the current and previous fieldwork, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor (discussed in Subsection 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 structures. 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 applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ 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 Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the underground parking level 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 5 on the next page. The flexible pavement structure presented in Table 6 and Table 7 should be used for driveways and car only parking areas and at grade access lanes and heavy loading parking areas.

Table 5 - Recommended Rigid Pavement Structure - Lower 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 – Hard to firm silty clay, or OPSS Granular A or OPSS Granular B Type II granular fill material placed over in situ soil.	

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 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.

Table 6 – Recommended Pavement Structure – Light Vehicle Parking	
Thickness (mm)	**Material Description**
40	**Wear Course –** Superpave 12.5-FC2 Asphaltic Concrete
50	**Binder Course –** Superpave 19.0 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
300	**SUBBASE –** OPSS Granular B Type II Crushed Stone
SUBGRADE - Either fill, in situ silty clay or sand/crushed stone material placed over in situ soil	

Table 7 – Recommended Pavement Structure –Local Roadways, Access Lanes and Heavy Vehicle Parking

Thickness (mm)	Material Description
40	**Wear Course –** Superpave 12.5-FC2 Asphaltic Concrete
50	**Upper Binder Course –** Superpave 19.0 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
400	**SUBBASE –** OPSS Granular B Type II Crushed Stone
SUBGRADE - Either fill, in situ silty clay or sand/crushed stone material placed over in situ soil	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable compaction equipment.

Pavement Structure Drainage

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

Where silty clay is encountered at subgrade level, consideration should be given to installing subdrains during the pavement construction. These drains should be constructed according to City of Ottawa specifications. The drains should be connected to a positive outlet. The subgrade surface should be crowned to promote water flow to the drainage lines. The subdrains will help drain the pavement structure, especially in early Spring when the subgrade is saturated and weaker and, therefore, more susceptible to permanent deformation.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is understood that the proposed building will accommodate three underground parking levels and will partially be founded on bedrock and/or dense glacial till. It is recommended that a dual-shoring (shoring/foundation) system such as a diaphragm wall, slurry wall or secant pile wall be considered for the building. As such, the shoring system is expected to reduce the potential for groundwater infiltration into the underground parking level structures. Based on Paterson's experience in the Ottawa area with slurry wall shoring systems, minor water infiltration may be observed long-term within the underground parking level structures. Therefore, to mitigate long-term water ingress into the underground parking levels, it is recommended to install an adequate foundation drainage system and negative side waterproofing. Refer to Figure 2 – Foundation Drainage System, for specific details of the foundation drainage recommendations in Appendix 1.

Alternatively, a watertight shoring system should be considered to avoid dewatering and settlement of the neighbouring properties.

Foundation Drainage System

A foundation drainage system is recommended for the underground parking levels to prevent water from seeping through the slurry walls.

Furthermore, to manage and control the groundwater infiltration to the building's storm sump pump(s) over the long-term, the following foundation drainage system is recommended to be installed on the interior side of the slurry walls or secant piles using the below methodology:

- ❑ Any discontinuities, leaks or imperfection in the foundation wall should be repaired and covered with a negative side waterproofing. While total application might not be required provision for installing a membrane such as Hygrothane by Elastochem or CN2000 series by Kelso should be considered.

- ❑ It is recommended that a composite foundation drainage membrane, such as 6000 series membrane by DeltaDrain, G100N by MiraDrain or equivalent approved other, be placed on the interior slurry wall face. The composite foundation drainage board should extend from finished grade to the footing level with the geotextile layer facing the prepped substrate surface (foundation wall). It is highly recommended that the drainage board be installed horizontally, in a shingle-fashion, with a minimum overlapping of 150 mm between the sheets to minimize seams throughout the system. The drainage should drain down to the lower slab and subfloor drainage system. Sleeves or continuous drainage will be required between the different parking levels.

- It is recommended that a 100-150 mm thick 35 MPa shotcrete liner be installed overlying the recommended drainage system to further prevent seepage into the underground parking levels. The shotcrete layer will provide an aesthetic finish to the interior underground parking levels and adequately seal the proposed drainage system.

- Furthermore, it is recommended that 150 mm diameter sleeves placed at 3 m centres be cast in the proposed shotcrete liner to allow water to flow to an interior underfloor drainage system. The underfloor drainage system should direct water to the storm sump pit(s) within the lower basement area.

Interior Underfloor Drainage System

The interior underfloor drainage system will be required to control water infiltration below the lowest underground parking level slab and to redirect groundwater from the buildings foundation drainage system to the buildings sump pit(s). The interior underfloor drainage pipes should consist of a 150 mm diameter corrugated perforated PVC pipes surrounded by a minimum of 150 mm of 19 mm clear crushed stone. It is recommended that the interior underfloor drainage system be mechanically connected to the 150 mm drainage sleeves and gravity connected to the underfloor drainage system which in turn is connected to the buildings storm sump pit(s).

The final spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Elevator Pit Waterproofing

The horizontally applied Colphene BSW H waterproofing membrane (or approved other) should be placed on an adequately prepared mud slab and extend vertically within the inside of the temporary forms of the elevator raft slab. Once the concrete raft slab and elevator shaft sidewalls are poured in place, it is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved other) should be applied to the exterior of the elevator pit sidewalls. The Colphene Torch'n Stick waterproofing membrane should extend over the vertical portion of the previously applied Colphene BSW H waterproofing membrane installed on the concrete raft slab in accordance with the manufacturer's specifications. As a secondary defense, a continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the concrete raft slab below the elevator pit sidewalls.

A protection board should be placed over the waterproofing membrane to protect the waterproofing membrane from damage during backfilling operations. The area between the elevator pit and bedrock excavation face should be in-filled with lean concrete, OPSS Granular B Type 2 or Granular A crushed stone. Refer to Figure 3 – Waterproofing System for Elevator, for specific details of the elevator waterproofing in Appendix 2.

Foundation Backfill

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials.

The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

Adverse Effects from Dewatering on Adjacent Structures

Since the excavation is expected to extend in a water bearing sandy till, construction dewatering is not recommended at depths greater than 6 to 7 m. The excavation should consider the use of a nearly waterproofed shoring system. It is estimated that groundwater lowering will affect the residential neighborhood to the north if more than 400,000 L/day is pumped during the excavation process. The use of a secant or diaphragm wall socketed a minimum of 1.5 m in bedrock or below excavation level (if extended into rock) will lower the groundwater infiltration into the excavation to controllable and acceptable levels.

The temporary dewatering of the bedrock during the excavation and construction stage will not be susceptible to significant consolidation of the material.

Implementation of dual use shoring system recommended above is expected to limit the drawdown of the local groundwater table over the long term and in a limited area. Therefore, in our opinion, no adverse effects to nearby structures and infrastructure are expected over the long term if a watertight shoring is used for construction.

6.2 Protection of Footings Against Frost Action

Perimeter footings, of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard. A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other exterior unheated footings.

However, foundations which are founded directly on clean, surface-sounded bedrock with no cracks or fissures, and which is approved by Paterson at the time of construction, is not considered frost susceptible and does not require soil cover.

6.3 Excavation Side Slopes

Temporary 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.

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 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.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer.

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

The temporary system could consist of diaphragm walls or secant pile walls. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced.

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

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

Table 8 - 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 are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

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

Diaphragm Wall System

For design purposes, the earth pressure acting on a slurry wall shoring system can be estimated using a trapezoidal earth pressure envelope with a maximum pressure of 0.3·γ·H for strutted or anchored shoring. The earth pressure will be zero for the top and bottom of the excavation and will increase to the maximum which occurs at 0.25·H from both the bottom and the top of the excavation. The earth pressure distribution can also be estimated using an earth pressure coefficient and a quasi-hydrostatic distribution.

The active earth pressure coefficient should be used where wall movements are permissible while the at-rest pressure coefficient should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations (i.e. below the groundwater level).

The excavation of a diaphragm wall should be carried out in sections or panels with the excavation filled with a bentonite-rich slurry to provide adequate support for the trench walls. Once the excavation is complete, reinforcing may be installed, if required, and concrete can be poured from the bottom of the excavation using tremie methods.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 95% of the material's SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce the potential 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 SPMDD.

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 compatible brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be low through the sides of the excavation and controllable using open sumps if a watertight shoring system is used. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

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

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

If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

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

The subsurface conditions mostly consist of frost susceptible materials. In the presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and tarpaulins or other suitable means.

The base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations 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. Precaution must be taken where excavations are carried out in close proximity of existing structures, which may be adversely affected due to the freezing conditions.

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 samples 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 severe to a very aggressive environment.

7.0 Recommendations

For the foundation design data provided herein to be applicable that a material testing and observation services program is required to be completed. The following aspects be performed by the geotechnical consultant:

- ❑ Review of the site master grading plan, once available.
- ❑ Review of the excavation and shoring plan (can be prepared by Paterson)
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Observation of the placement of the foundation insulation, if applicable.
- ❑ Observe and review the installation of the drainage and waterproofing system.
- ❑ 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 and follow-up field density tests to determine the level of compaction achieved.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

The recommendations provided in this report are intended for the use of design professionals associated with this project. Contractors bidding on or undertaking the work should examine the factual information contained in this report and the site conditions, satisfy themselves as to the adequacy of the information provided for construction purposes, supplement the factual information if required, and develop their own interpretation of the factual information based on both their and their subcontractor's construction methods, equipment capabilities and schedules.

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 Brigil Construction or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
September 6, 2024
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Pratheep Thirumoolan, M.Eng. Joey R. Villeneuve, M.A.Sc., P.Eng, ing.

Report Distribution:

- ❑ Brigil Construction
- ❑ Paterson Group Inc

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use High-Rise Development
1299 Richmond Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** March 13, 2023

FILE NO. PG6598

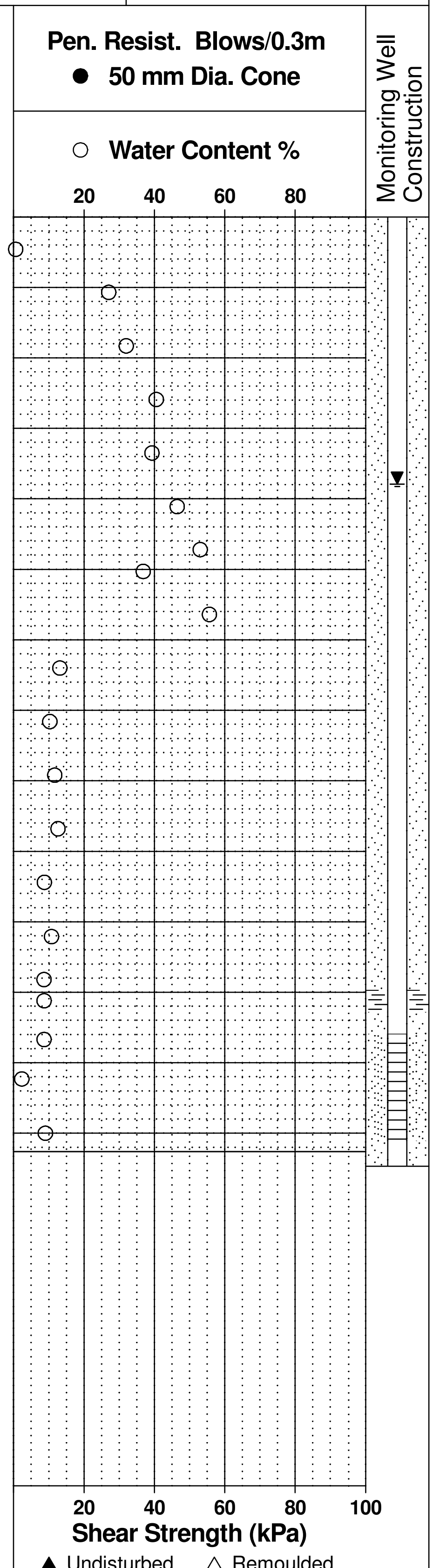
HOLE NO. BH 1-23

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.32
Asphaltic concrete	0.13						
FILL: Grey crushed stone, some sand	0.69	AU	1				
Loose, brown **SANDY SILT,** trace to some clay		SS	2	100	9	1	68.32
		SS	3	100	5	2	67.32
	2.97	SS	4	100	5		
Very stiff to stiff, brown **SILTY CLAY,** trace sand seams		SS	5	100	3	3	66.32
		SS	6	100	5	4	65.32
		SS	7	100	2	5	64.32
	6.02	SS	8	100	1	6	63.32
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders		SS	9	25	5		
		SS	10	100	3	7	62.32
		SS	11	17	4	8	61.32
		G	12				
		SS	13	33	3	9	60.32
		SS	14		3	10	59.32
	11.13	SS	15	100	4	11	58.32
GLACIAL TILL: Dense to very dense, brown silty sand to sandy silt, some clay, gravel, cobbles and boulders		SS	16	84	40		
		S	17	67	50+	12	57.32
	13.26	SS	18	50	50+	13	56.32

End of Borehole

Practical refusal to augering at 13.26m depth.

(GWL @ 3.79m - March 23, 2023)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use High-Rise Development
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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE March 13, 2023

FILE NO. PG6598

HOLE NO. BH 2-23

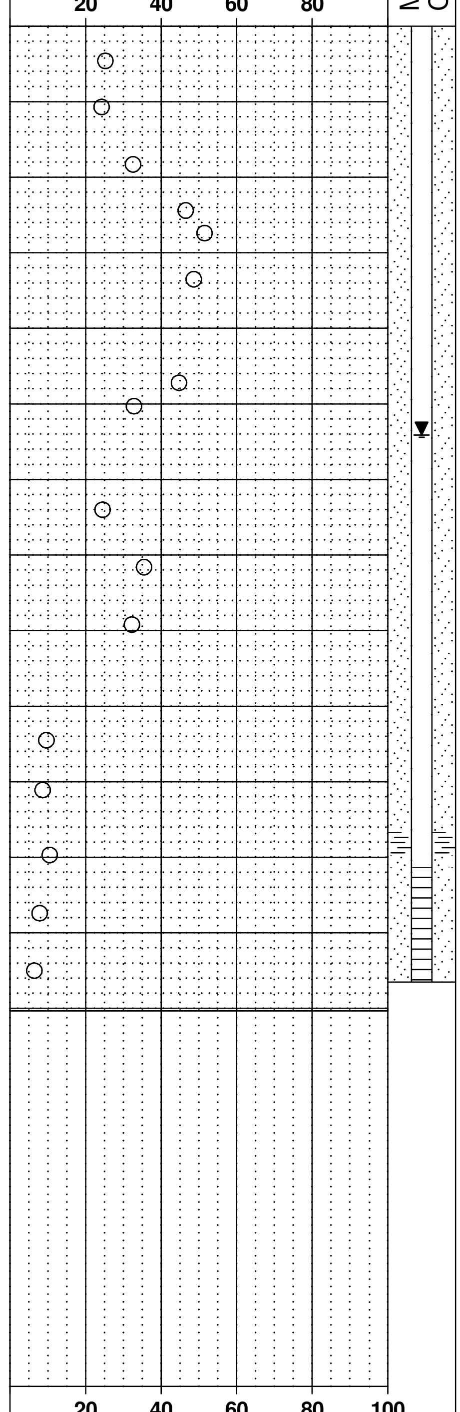
SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.06
Asphaltic concrete	0.08						
FILL: Crushed stone, some silty sand	0.23	AU	1				
FILL: Brown silty clay, some sand	0.76	SS	2	100	5	1	68.06
Loose, brown **SANDY SILT,** trace to some clay	2.21	SS	3	100	6	2	67.06
Very stiff to stiff, brown **SILTY CLAY,** some sand seams		SS	4	100	3		
		SS	5	100	2	3	66.06
						4	65.06
- stiff to firm and grey by 4.7m depth		SS	6	100	2	5	64.06
						6	63.06
		SS	7	100	1		
		SS	8	100	1	7	62.06
	8.23	SS	9	100	1	8	61.06
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	9.30	SS	10	0	7	9	60.06
GLACIAL TILL: Compact to dense, grey silty sand to sandy silt with gravel, cobbles and boulders		SS	11	58	24		
		SS	12	88	50+	10	59.06
		SS	13	29	50+	11	58.06
		SS	14		50+	12	57.06
		SS	15	71	50+		
	13.03	SS	16	0	50+	13	56.06

End of Borehole

Practical refusal to augering at 13.03m depth.

(GWL @ 5.41m - March 23, 2023)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use High-Rise Development
1299 Richmond Road, Ottawa, Ontario

DATUM Geodetic

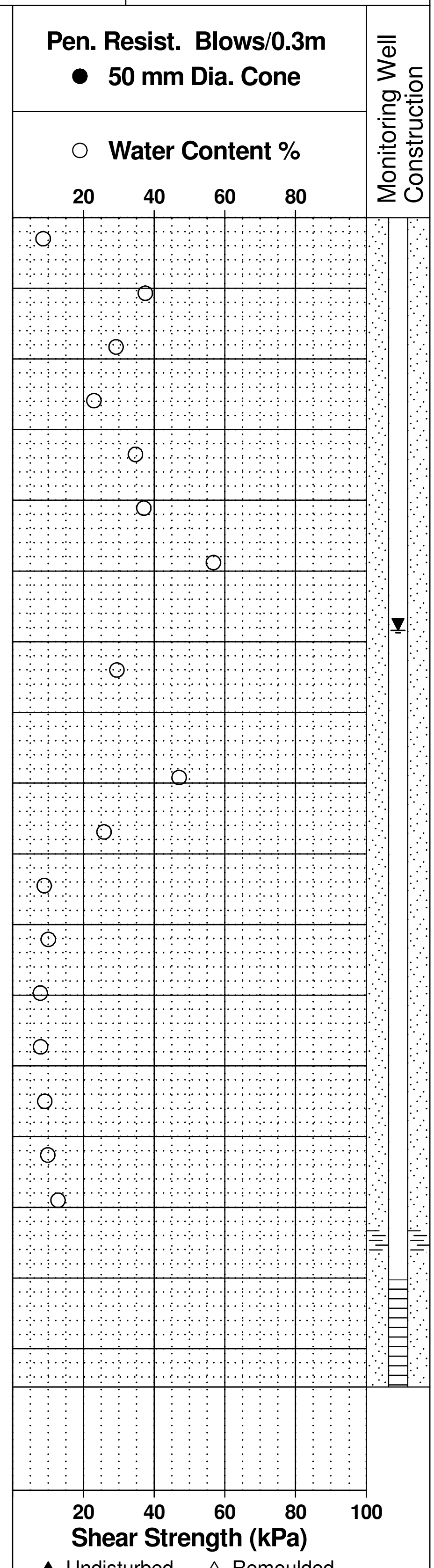
REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** March 14, 2023

FILE NO. PG6598

HOLE NO. BH 3-23

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.30
Asphaltic concrete	0.05	AU	1				
FILL: Crushed stone	0.56						
Loose, brown **SANDY SILT,** trace to some clay		SS	2	100	6	1	68.30
- some sand seams by 2.3m depth		SS	3	92	4	2	67.30
	2.74	SS	4	100	7		
Very stiff, brown **SILTY CLAY** some sand seams		SS	5	100	4	3	66.30
		SS	6	100	2	4	65.30
- silty sand layer from 5 to 5.2m depth		SS	7	100	5	5	64.30
						6	63.30
		SS	8	100	1	7	62.30
- trace gravel by 8.1m depth		SS	9	100	1	8	61.30
	8.99	SS	10	8	2		
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders		SS	11	8	3	9	60.30
	10.21	SS	12	21	16	10	59.30
GLACIAL TILL: Compact to very dense, grey silty sand, some gravel, cobbles and boulders, trace clay		SS	13	100	50+	11	58.30
		SS	14	67	50+	12	57.30
		SS	15	83	50+		
		SS	16	100	50+	13	56.30
	14.05	SS	17	86	50+	14	55.30
BEDROCK: Excellent quality, grey quartz sandstone		RC	1	100	100	15	54.30
	16.54	RC	2	100	98	16	53.30
End of Borehole							
(GWL @ 5.84m - March 23, 2023)							



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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE March 15, 2023

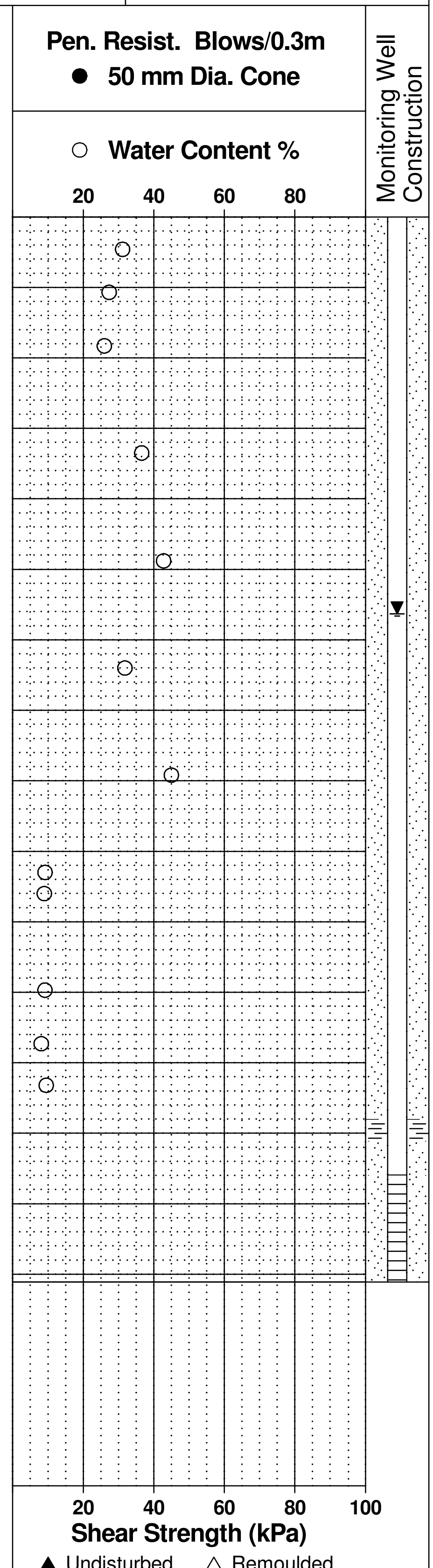
FILE NO. PG6598

HOLE NO. BH 4-23

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08	AU	1		
FILL: Crushed stone with silty sand	0.38				
FILL: Brown silty clay, trace sand and gravel	0.69	SS	2	100	6
Loose, brown **SANDY SILT,** trace to some clay	3.05	SS	3	100	6
Very stiff, brown **SILTY CLAY,** trace to some sand seams	5.49	SS	4	100	4
		SS	5	100	6
Stiff to firm, grey **SILTY CLAY**	8.53	SS	6	100	1
		SS	7	100	1
GLACIAL TILL: Grey silty clay with sand and gravel	9.45				
GLACIAL TILL: Very dense, grey silty sand to sandy silt with gravel, cobbles and boulders	12.44	SS	8	100	50+
		SS	9	58	50+
		SS	10	58	50+
		SS	11	90	50+
BEDROCK: Excellent quality, grey quartz sandstone	15.11	RC	1	100	100
		RC	2	100	95
		RC	3	100	95
End of Borehole					

(GWL @ 5.63m - March 23, 2023)

DEPTH (m)	ELEV. (m)
0	69.18
1	68.18
2	67.18
3	66.18
4	65.18
5	64.18
6	63.18
7	62.18
8	61.18
9	60.18
10	59.18
11	58.18
12	57.18
13	56.18
14	55.18
15	54.18



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use High-Rise Development
1299 Richmond Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE March 15, 2023

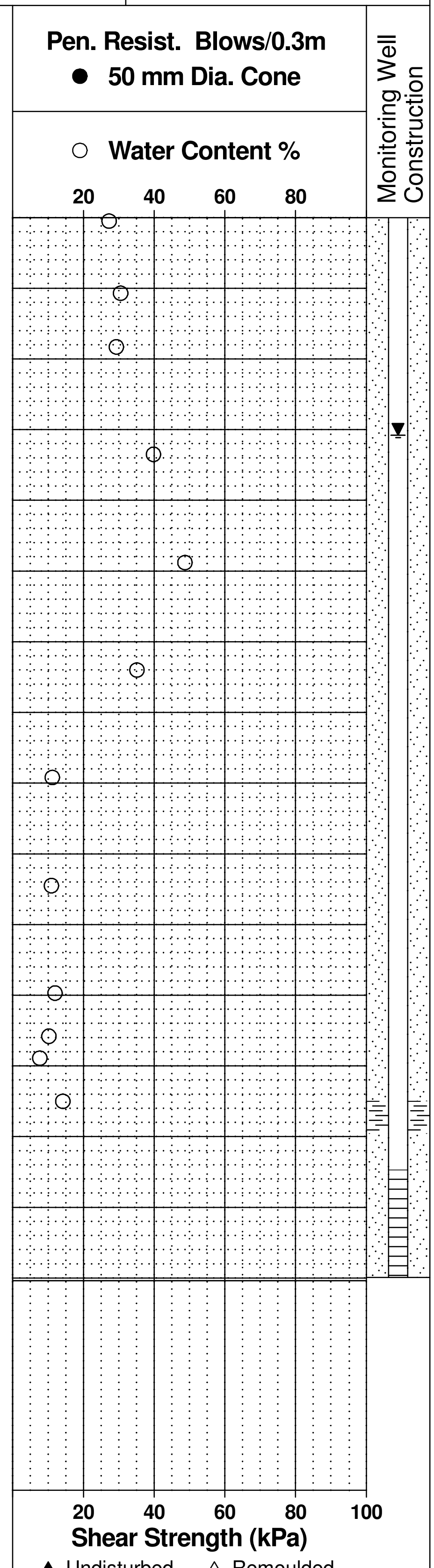
FILE NO. PG6598

HOLE NO. BH 5-23

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.10	AU	1		
FILL: Brown sandy silt, some clay, trace gravel and organics	0.60	SS	2	83	11
Compact to loose, brown **SANDY SILT,** trace to some clay; - increasing clay content with depth	2.59	SS	3	92	7
Hard to very stiff, brown **SILTY CLAY,** some sand; - stiff and grey by 5.5m depth	6.10	SS	4	100	5
		SS	5	100	3
GLACIAL TILL: Grey silty clay with sand and gravel, occasional cobbles	11.99	SS	6	75	3
		SS	7	58	12
		SS	8	83	4
		SS	9	58	2
		SS	10	100	1
GLACIAL TILL: Dense, grey silty sand to sandy silt with gravel, cobbles and boulders, some to trace clay	12.78	SS	11	83	45
BEDROCK: Excellent quality, grey quartz sandstone	15.04	RC	1	100	100
		RC	2	100	98
End of Borehole					

(GWL @ 3.08m - March 23, 2023)

DEPTH (m)	ELEV. (m)
0	69.55
1	68.55
2	67.55
3	66.55
4	65.55
5	64.55
6	63.55
7	62.55
8	61.55
9	60.55
10	59.55
11	58.55
12	57.55
13	56.55
14	55.55
15	54.55



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

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

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu	-	Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: 1 < Cc < 3 and Cu > 4
Well-graded sands have: 1 < Cc < 3 and Cu > 6
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

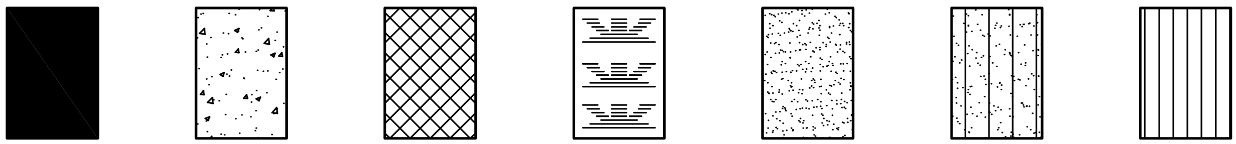
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

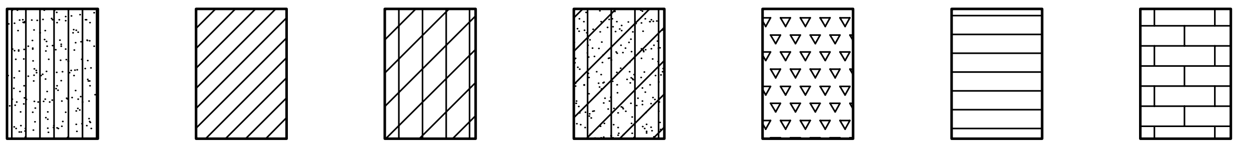
k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

SYMBOLS AND TERMS (continued)

STRATA PLOT



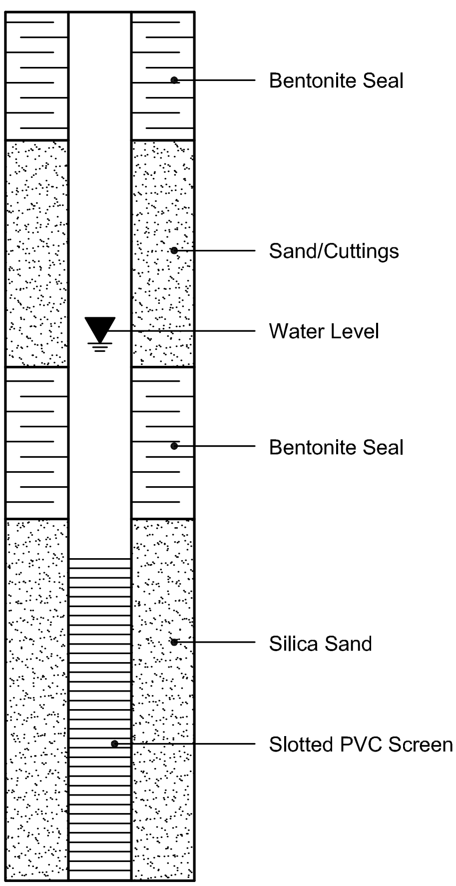
Topsoil | Asphalt | Fill | Peat | Sand | Silty Sand | Silt



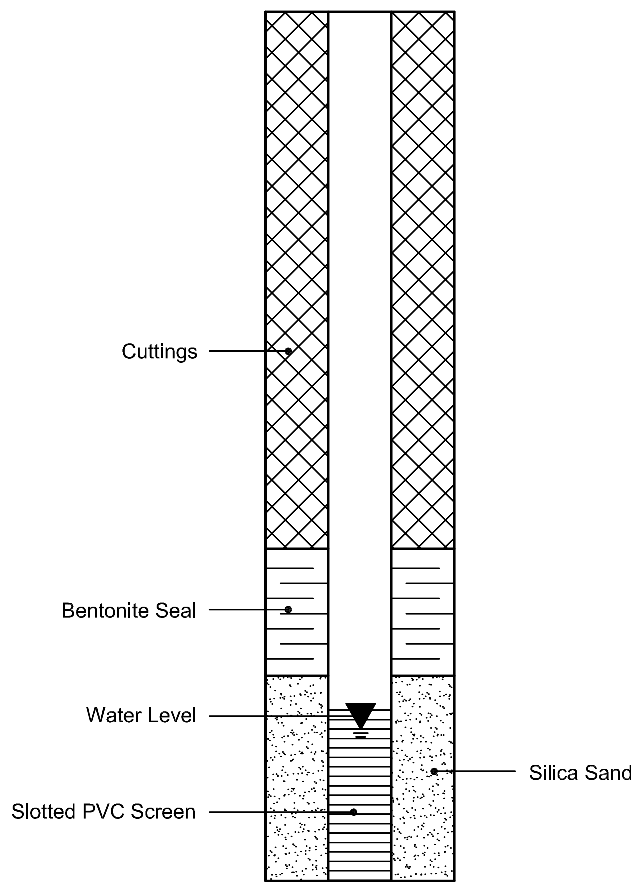
Sandy Silt | Clay | Silty Clay | Clayey Silty Sand | Glacial Till | Shale | Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 57016

Report Date: 20-Mar-2023

Order Date: 15-Mar-2023

Project Description: PG6598

	Client ID:	BH3-23 / SS4	-	-	-
	Sample Date:	14-Mar-23 09:00	-	-	-
	Sample ID:	2311337-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	81.3	-	-	-
General Inorganics					
pH	0.05 pH Units	7.46	-	-	-
Resistivity	0.1 Ohm.m	5.5	-	-	-
Anions					
Chloride	10 ug/g dry	1200	-	-	-
Sulphate	10 ug/g dry	128	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 – FOUNDATION DRAINAGE SYSTEM

FIGURE 3 – ELEVATOR PIT WATERPROOFING

DRAWING PG6598-1 - TEST HOLE LOCATION PLAN

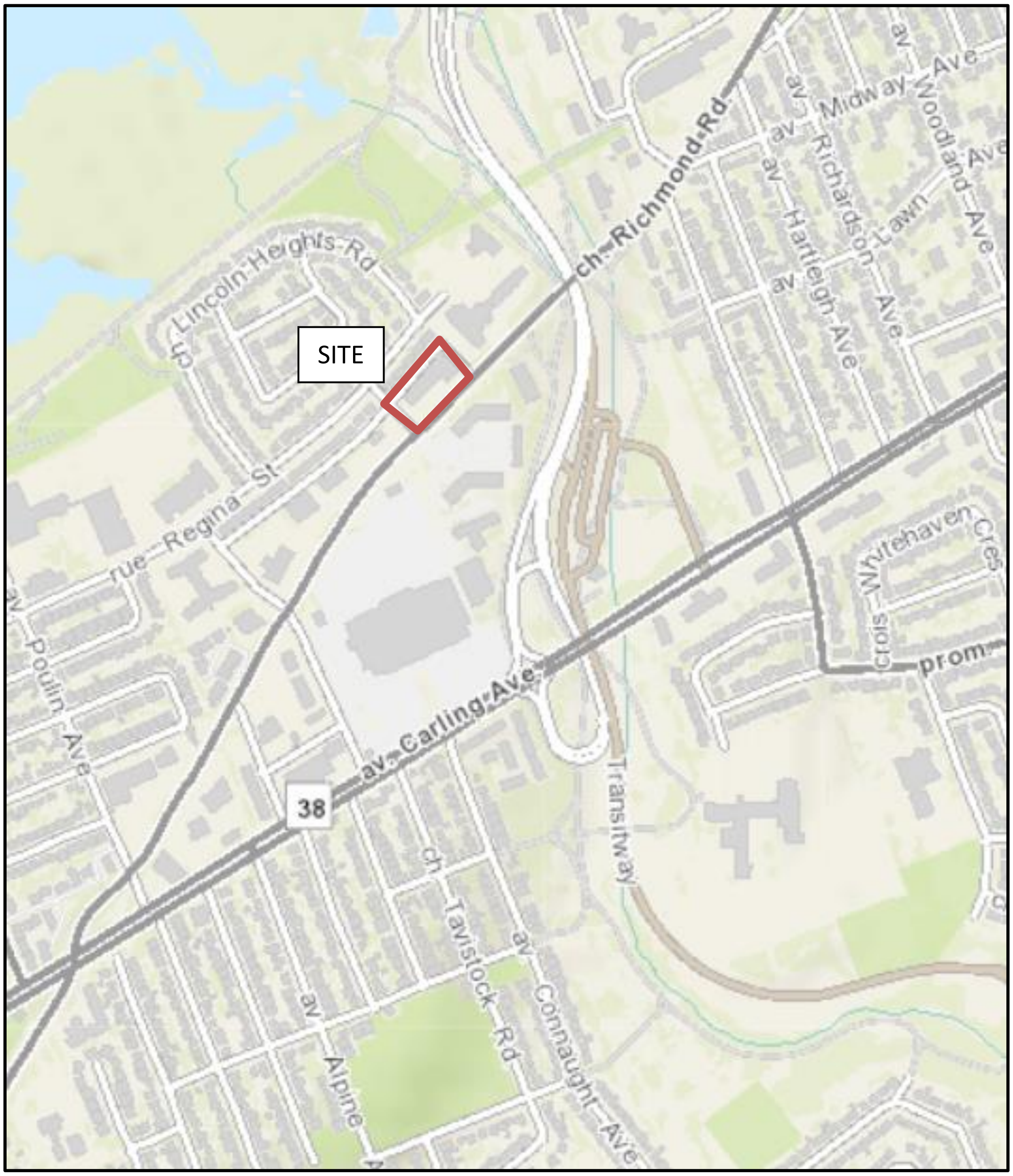
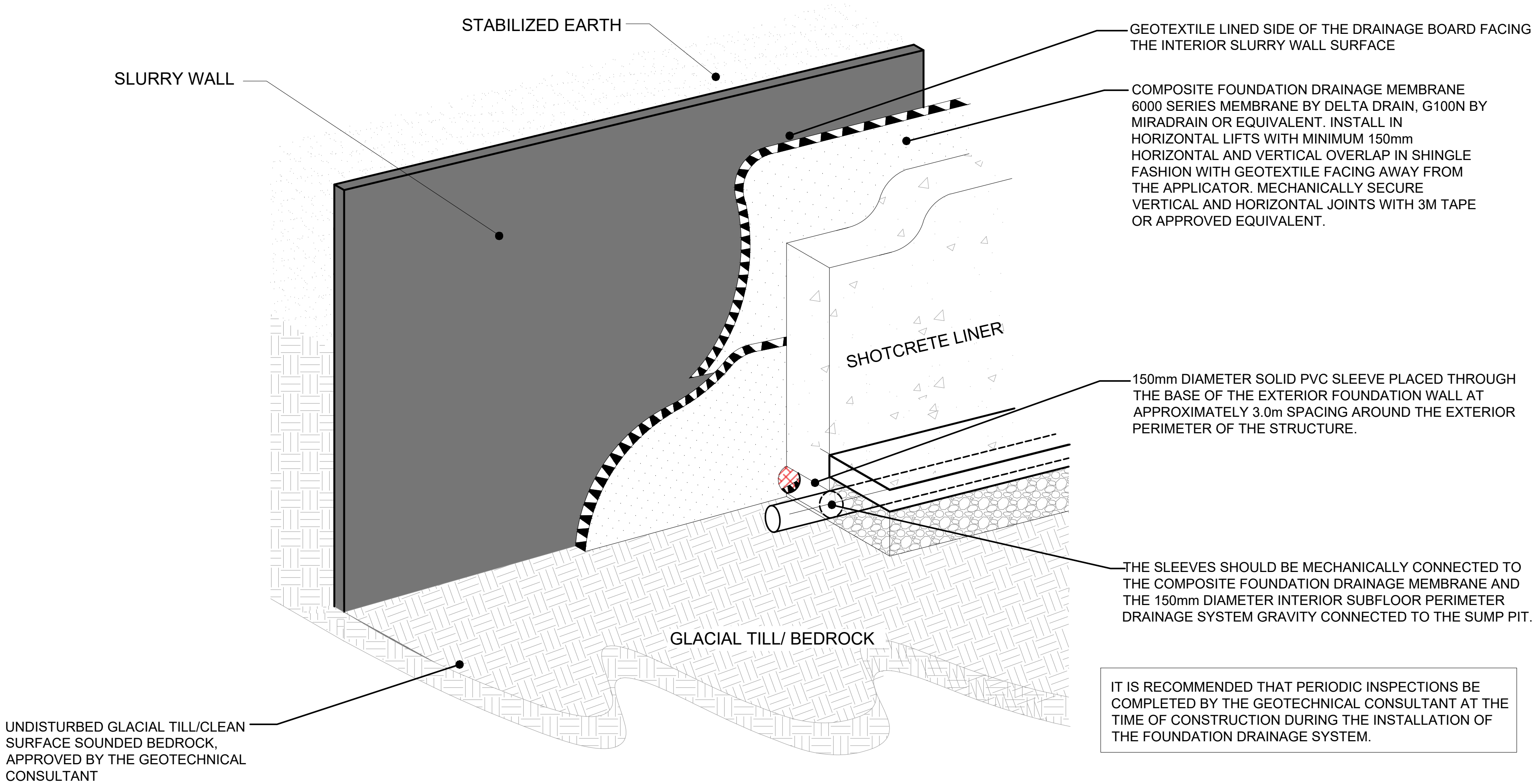


FIGURE 1

KEY PLAN

PATERSON GROUP



IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE FOUNDATION DRAINAGE SYSTEM.

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

BIRGIL

GEOTECHNICAL INVESTIGATION

PROPOSED MIX-USE HI-RISE DEVELOPMENT

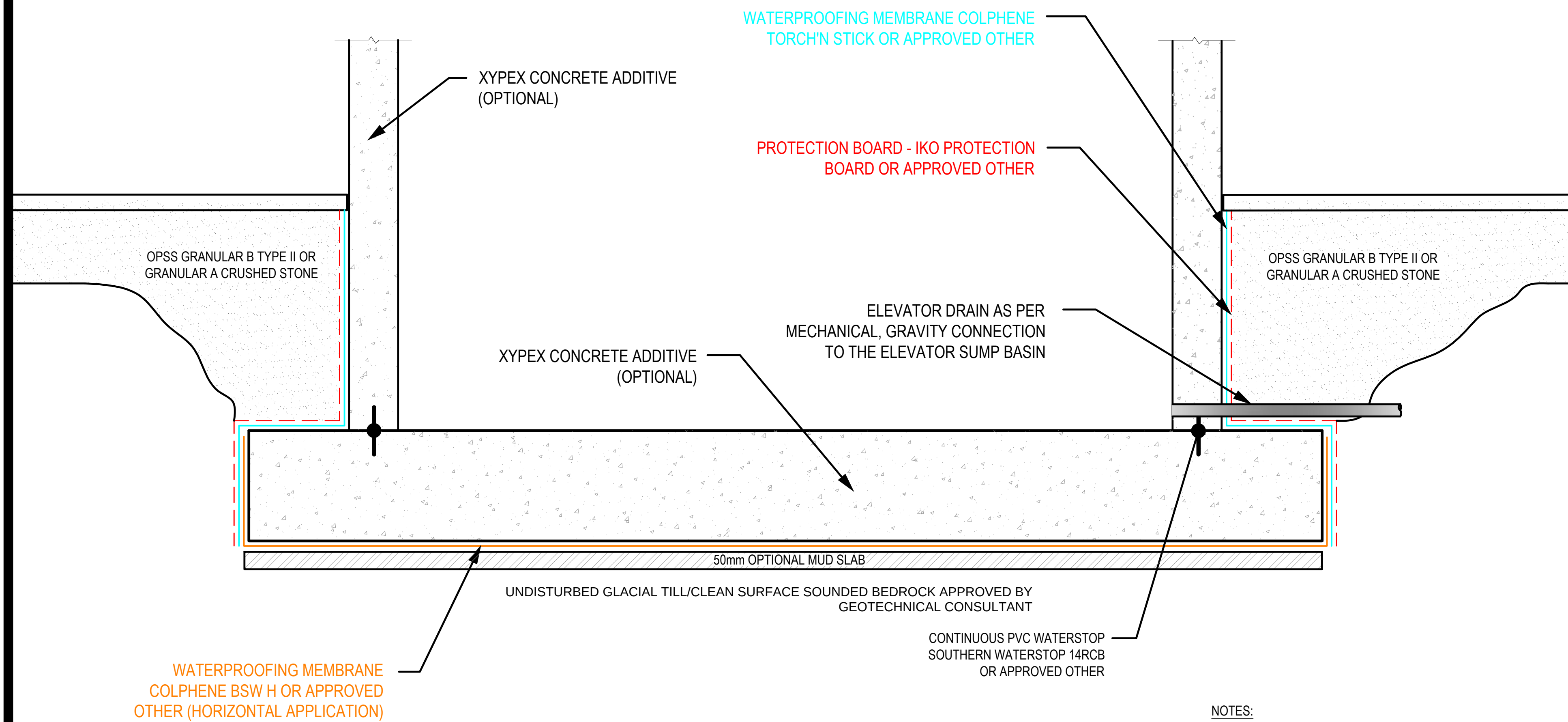
1299 RICHMOND ROAD

OTTAWA, ONTARIO

Title:

FOUNDATION DRAINAGE SYSTEM

Scale: NTS	Date: 04/2023
Drawn by: YA	Report No.: PG6598-1
Checked by: PT	Drawing No.: FIG. 2
Approved by: JV	Revision No.:



NOTES:

1. IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE ELEVATOR WATERPROOFING MEMBRANE(S).

PATERSON GROUP

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

BIRGIL

GEOTECHNICAL INVESTIGATION

PROPOSED MIX-USE HI-RISE DEVELOPMENT

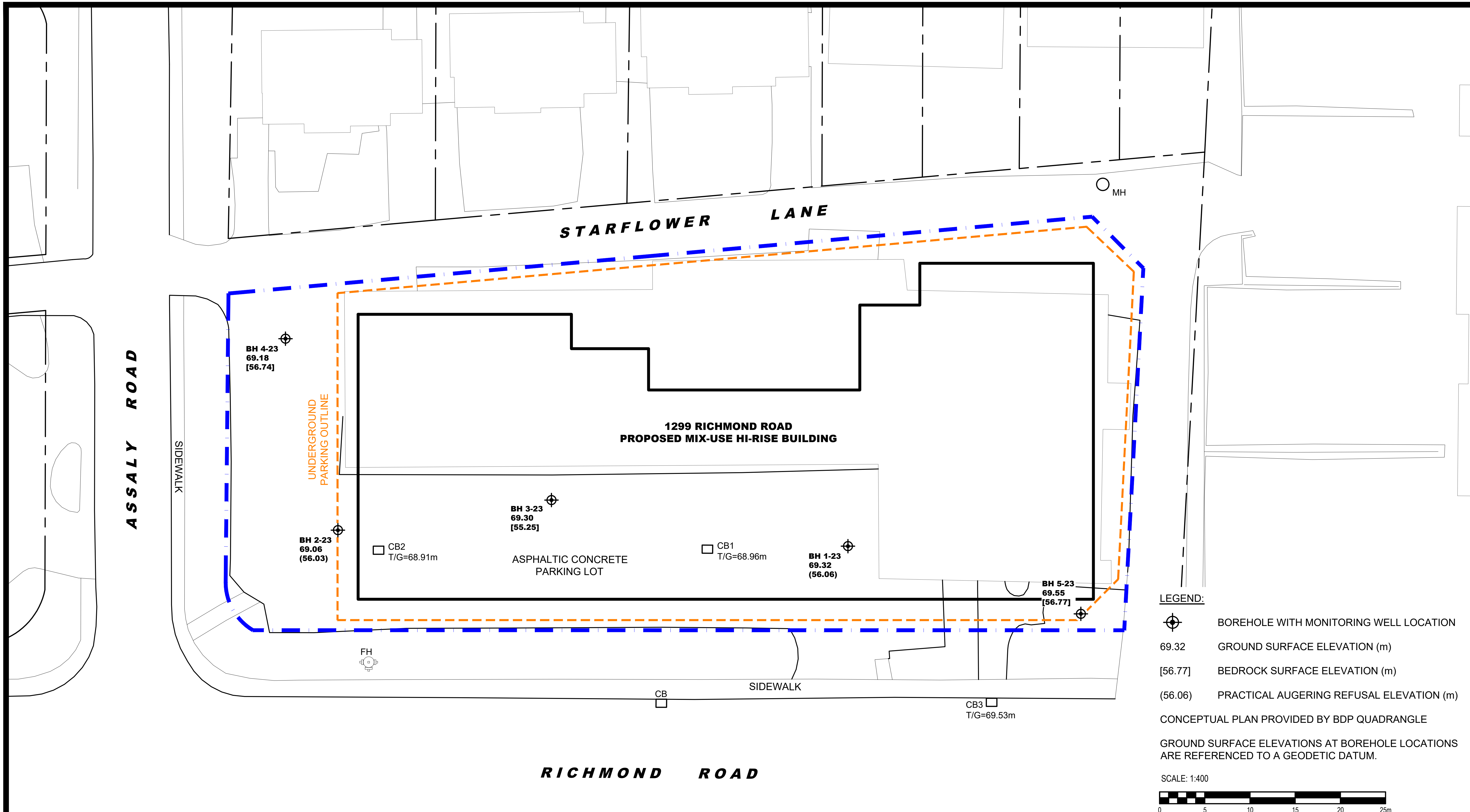
1299 RICHMOND ROAD

OTTAWA, ONTARIO

Title:

WATERPROOFING SYSTEM FOR ELEVATOR

Scale: N.T.S.	**Date:** 04/2023
Drawn by: YA	**Report No.:** PG6598-1
Checked by: PT	**Drawing No.:** FIGURE 3
Approved by: JV	**Revision No.:**



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

NO.	REVISIONS	DATE	INITIAL

BIRGIL

GEOTECHNICAL INVESTIGATION
PROPOSED MIX-USE HI-RISE DEVELOPMENT
1299 RICHMOND ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale:	1:400	Date:	03/2023
Drawn by:	YA	Report No.:	PG6598-1
Checked by:	NS	Dwg. No.:	PG6598-1
Approved by:	JV	Revision No.:	

11x17

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APPENDIX 3

TYPICAL FOUNDATION SLEEVE INSTALLATION

Photo 1 – Step 1: It is recommended that the upper 1/3 of the 150 mm drainage sleeve be cut at a 45 degree angle to hydraulically connect the composite foundation drainage board to the interior and underfloor drainage system.

Photo 2 – Step 2: It is recommended that the 150 mm diameter drainage sleeve be installed by carefully cutting an 'X' shaped incision through the composite foundation drainage and inserting the 150 mm diameter drainage sleeve inside the 'X' by pulling the four (4) triangular flaps towards the installer.

Photo 3 – Step 3: Apply a suitable primer prior to the placement of the adhesive tape such as 3M tape, WP200 BlueSkine or equivalent.

Photo 4 – Step 4: An adhesive such as 3M tape, BlueSkin, or equivalent be utilized to seal the 150 mm drainage sleeve to the composite foundation drainage board to act as a barrier in preventing concrete from blocking connection during the placement of the exterior concrete foundation wall.

Photo 5 – Step 5: As an additional precaution, it is also recommended that an adhesive tape be placed on the interior outlet end of the drainage sleeve between the temporary form work to further prevent concrete from entering the drainage sleeve during the placement of concrete. Once the temporary form work has been removed, the adhesive tape can be cut away to allow groundwater to have a positive gravity connection to the interior perimeter and underfloor drainage system.