Geotechnical Investigation

Proposed High-Rise Development

254 Argyle Avenue
Ottawa, Ontario

Azure Urban Developments Inc.

Report PG7026-1 Revision 1 dated April 15, 2024

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Azure Urban Developments Inc. to complete a geotechnical investigation for the proposed development to be located at 254 Argyle Avenue, Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the geotechnical investigation was to:

- determine the subsoil and groundwater conditions at the site by means of test holes.

- provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its 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 the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available conceptual drawings, it is understood that the proposed development will consist of a ten-storey building with one underground parking level. It is expected the proposed building's parking garage will extend throughout the majority of the subject site.

It is also understood that the existing church structure will be relocated in closer proximity to Argyle Avenue and be integrated into the proposed building. It is further 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 March 5 and 13, 2024, and consisted of advancing two (2) boreholes to a maximum depth of 13.7 m. and two (2) test pits that were excavated to 2.4 m below ground surface against the existing church structure. Previous investigations undertaken by Paterson consisted of advancing boreholes on November 25, 2019, to a maximum depth of 9.4 m below ground surface. The test holes were placed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations for the current investigation are presented on Drawing PG7026-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a low-clearance rubber-track drill rig operated by a two-person crew and the test pits were undertaken by the use of a rubber-tired backhoe excavating to the determined depths. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The testing procedure consisted of augering or excavating to the required depths and at the selected locations sampling the overburden.

Sampling and In Situ Testing

Soil samples were recovered during drilling from the auger flights or a 50 mm diameter split-spoon sampler and during test pitting from the test pit sidewalls. The split-spoon samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The split-spoon samples, auger grab- and grab samples recovered from the boreholes are shown as SS, AU, G, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

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

The thickness of the overburden was evaluated at BH 1-24 and BH 1 (2019 investigation) by a dynamic cone penetration test (DCPT). The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater

A monitoring well was installed in BH1A-24 to permit monitoring of the groundwater levels subsequent to the completion of the field investigations. Piezometers installed in previous borehole are no longer present throughout the subject site.

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

Sample Storage

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

3.2 Field Survey

The test hole locations were selected to provide general coverage of the subject site. The test hole location and ground surface elevation at the test holes for the current investigation were surveyed by Paterson using a high precision, handheld GPS and referenced to a geodetic datum. The location of the test holes are presented in Drawing PG7026-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. A total of one Atterberg limit test was completed on a select clay soil sample. Moisture content testing was completed on all retrieved soil samples recovered from the current invesigiaton.

The results of the testing are discussed in Subsection 4.2 and presented in Appendix 1 of this report.

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 by others. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an existing church located on the southern half of the site, with a walkway and landscaped areas on the northern half of the site fronting Argyle Avenue. The site is bordered by residential properties to the east, west, and south, and by Argyle Avenue to the north. The existing ground surface across the site is relatively level at an approximate geodetic elevation of 69.5 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consisted of either topsoil underlain by fill which is further underlain by a deposit of silty clay and then a deposit of glacial till.

Topsoil was encountered at BH 1-24, BH 1, BH 2, TP 1-24 and TP 2-24 and was observed to be approximately 750, 230, 300 and 400 mm thick, respectively.

Fill was encountered at the test holes and was observed to consist of a blend of brown silty sand and silty clay with variable amounts of gravel and topsoil up to depths ranging between 1.5 to 1.8 m below ground surface.

The fill layers were generally observed to be underlain by a deposit of silty clay which consisted of a layer of desiccated very stiff to stiff brown clay crust. The brown silty clay layer was observed at depths ranging between 1.8 to 3.7 m below the ground surface. The brown silty clay was observed to be underlain by a layer of unweathered, firm to stiff grey silty clay which was observed to extend to 12.8 m below the existing ground surface.

The glacial till was encountered below the clay deposit and observed to be compact to dense. The glacial till soil matrix comprised silty clay to silty sand with variable amounts of clay, sand, gravel, cobbles, and boulders and it was observed that the clay content was decreasing with depth. The glacial till was observed encountered at BH 1-24 at a depth of 12.8 m below the existing ground surface.

Practical refusal to the DCPT was encountered at a depth of 19.79 m below the existing ground surface at BH 1-24.

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

Atterberg Limit and Shrinkage Tests

Atterberg limits testing, as well as associated moisture content testing, was completed on recovered silty clay samples at selected locations throughout the subject site. The results of the Atterberg limits are presented in Table 1 below.

Table 1 – Summary of Atterberg Limits Tests

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	Classification
BH 1-24 SS5	3.35	87	27	60	CH

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; CH: Inorganic Clay of High Plasticity

Bedrock

Based on available geological mapping, the site is located in an area where the bedrock consists of shale of the Billings formation with a drift thickness of 15 to 25 m.

Existing Building Foundation

Two test pits were advanced against the existing church structure to confirm the founding depth of the structure and overall foundation wall assembly. The foundation wall was generally observed to consist of damp-proofed concrete and backfilled against by fill containing variable amounts of clay, silt, sand, gravel and inorganic debris. The top of the footing was encountered at an approximate elevation of 68.45 and 68.34 m at TP 1-24 and TP 2-24, respectively. The underside of footing was encountered at an elevation of 68.15 m and 68.04 m at TP 1-24 and TP 2-24 respectively.

It should be noted that it our understanding that the southern portion of the foundation for the church structure had been supplemented by the use of end-bearing micro-piles. However, the test pits were not undertaken throughout the area of the previously installed micro-piles.

4.3 Groundwater

Groundwater levels were measured in the installed monitoring well during the current investigation. The measured groundwater level (GWL) readings are presented in Table 2 below and are shown on the Soil Profile and Test Data sheets in Appendix 1.

Table 2 – Summary of Groundwater Levels				
Test Hole	**Ground Surface Elevation (m)**	**Measured Groundwater Level**		**Date Recorded**
		Depth (m)	**Elevation (m)**	
BH 1A-24	69.42	2.95	66.47	March 27, 2024
BH 1	69.41	6.08	63.33	November 29, 2019
BH 2	69.47	3.84	65.63	November 29, 2019
Note: The ground surface elevation was surveyed using a handheld GPS and referenced to a geodetic datum.				

It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole. The groundwater table can also be estimated based on recovered soils samples moisture levels, soil sample coloring and consistency. Based on this methodology, the groundwater table is estimated to be at **3 to 4 m** depth below the existing grade.

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

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. The proposed multi-storey building is recommended to be founded on a raft foundation placed on an undisturbed stiff silty clay bearing surface or a deep foundation, such as end-bearing piles, extending to the bedrock surface.

Due to the presence of a silty clay layer, the proposed development will be subjected to grade raise restrictions. Our permissible grade raise recommendations are discussed in Subsection 5.3.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. However, the site excavation is expected to occupy the majority of the site to a depth significantly below the existing grade, therefore, all topsoil and fill materials will be removed from within the perimeter of the proposed building.

Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. The tops of previous pile foundation structures are recommended to be cut down a minimum of 300 mm below the underside of any proposed foundation structure and reinstated using engineered fill, such as OPSS Granular A or OPSS Granular B Type II, compacted to a minimum of 95% of materials SPMDD. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Fill Placement

Fill used for grading beneath the proposed building should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. Granular material should be tested and approved prior to delivery to the site. The fill should be placed in loose lifts of 300 mm thick or less and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building area should be compacted to at least 98% of the Standard Proctor Maximum Dry Density (SPMDD).

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

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a geocomposite drainage membrane, such as CCW MiraDRAIN 2000 or Delta-Teraxx.

Vibration Considerations

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

Two parameters determine the recommended vibration limit, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards.

Protection of Subgrade (Raft Foundation)

Since the subgrade material for the building's foundation is expected to consist of firm to stiff, grey silty clay, it is recommended that a minimum 50 mm thick lean concrete mud slab (minimum 15 MPa 28-day compressive strength) be placed on the undisturbed silty clay subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic or workers and equipment.

The final excavation of the raft bearing surface level and the placing of the mud slab should be completed in smaller sections to avoid exposing large areas of the silty clay to potential disturbances due to drying. The bearing medium should be reviewed and approved by Paterson personnel prior to placing the mud slab layer.

Compacted Granular Fill Working Platform (Piled Foundation)

Should the proposed structure be supported on a pile foundation, the use of heavy equipment would be required to install the piles (i.e., pile driving crane). It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance to the underlying soil.

A typical working platform could consist of 600 mm of OPSS Granular B, Type II crushed stone placed and compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in maximum 300 mm thick lifts.

Once the piles have been driven and cut off, the working platform can be re-graded, and soil tracked in, or soil pumping up from the pile installation locations, can be bladed off and the surface can be topped up, if necessary, and re-compacted to act as the substrate for further fill placement for basement slab structure.

5.3 Foundation Design

Raft Foundation

Based on the expected loads from the proposed structure, a raft foundation bearing on the undisturbed stiff, grey silty clay bearing surface may be considered for foundation support for the proposed building.

For design purposes, it was assumed that the base of the raft foundation would be located at an approximate geodetic elevation of **63.0 to 64.0 m** and would be provided with one level of underground parking. If the raft is anticipated to be founded higher than an elevation of 64.0 m, Paterson must be notified to review the applicability of the following bearing resistance values.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. 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.

For the raft slab foundation, a bearing resistance value at SLS (contact pressure) of **200 kPa** will be considered acceptable for a raft supported on the undisturbed, stiff silty clay. The factored bearing resistance (contact pressure) at ULS can be taken as **300 kPa**. For this case, the modulus of subgrade reaction was calculated to be **7.4 MPa/m** for a contact pressure of **300 kPa**.

These values are only considered applicable for a raft foundation that would be founded between a geodetic elevation of 64.0 to 63.0 m. <u>If it is anticipated the raft would be founded higher than this elevation range, Paterson should be notified to review and advise on an appropriate raft foundation design contact pressure</u>.

The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS. Based on the following assumptions for the raft foundation, the high-rise portion of the proposed structure can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

Deep Foundation – End Bearing Piles

A deep foundation method, such as end bearing piles, may be considered for the foundation support of the proposed building should its loading exceed the load bearing capacity provided for a raft slab foundation. 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 3. 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 3 – 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	925	1100	9	27
245	11	1050	1250	9	31
245	13	1200	1400	9	35

Re-striking of all piles, at least once, will also be required after at least 48 hours have elapsed since initial driving. A full-time field review program should be conducted during the pile driving operations to record the pile lengths, ensure that the refusal criteria is met and that piles are driven within the location tolerances (within 75 mm of proper location and within 2% of vertical).

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.

Conventional Shallow Foundations (Auxiliary Structures)

The following conventional spread footing bearing resistance values may be considered only for portions of the underground parking garage structure located beyond the building footprint and other lightly loaded ancillary structures. These values are not considered applicable to the high-rise portion of the proposed building.

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over an undisturbed, very stiff brown silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**.

Strip footings, up to 2 m wide, and pad footings, up to 4 m wide, placed over an undisturbed, stiff grey silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **80 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **120 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS. It is recommended to carry provisions to provide a minimum 50 mm thick mud slab for all bearing surfaces that would be located upon a stiff, grey silty clay to limit disturbance by construction traffic and weather.

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 an undisturbed soil bearing surface and designed using the bearing resistance values at SLS provided above will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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 the encountered overburden material above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Permissible Grade Raise Restrictions

Based on the undrained shear strength values of the silty clay deposit encountered throughout the subject site and anticipating the structure to be supported by a raft slab foundation, a permissible grade raise restriction of **0.8 m** is recommended for settlement sensitive structures that will be located throughout the subject site.

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

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for foundations constructed at the subject site, according to Table 4.1.8.4.A of the 2012 Ontario Building Code (OBC 2012). The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

Where a raft slab is utilized, a granular layer of OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements. The recommended pavement structures noted in Subsection 5.7 will be applicable where the basement level underlying foundation support consists of a raft foundation.

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

A subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lowest basement floor. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed. This is discussed further in Section 6.1 of this report.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

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

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall.

The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case. Actual earth pressures could be higher than the "at-rest" case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

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

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

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2
The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to the 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 the OBC 2012.

5.7 Pavement Design

Pavement Structure Over Overburden

The following pavement structures may be considered for the access lane between the right-of-way and the access ramp as detailed in Tables 4, 5 and 6.

Table 4 – Recommended Hard Landscaping – Pedestrian Walkways	
Thickness (mm)	**Material Description**
Specified by Others	**Wear Course** – Interlocking Stones/Brick Pavers
25 - 40	**Levelling Course –** Stone Dust or Sand
300	**SUBBASE –** OPSS Granular A
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill.	

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

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

Pavement Structure Over Raft Foundations

Based on the concrete raft slab subgrade for the underground parking level, the pavement structure indicated in the following tables may be considered for design purposes:

Table 7 - Recommended Rigid Pavement Structure - Lower Level	
Thickness (mm)	**Material Description**
Specified by Others	**Rigid Concrete Pavement –** Class C2 Exposure Class Reinforced Concrete
Min. 300*	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE – Reinforced Concrete Raft Slab **NOTE "*"**: OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements, however, is recommended to be a minimum of 300 mm to support the proposed rigid pavement structure.	

Table 8 - Recommended Pavement Structure - Car-Only Parking Areas (Raft Slab)	
Thickness (mm)	**Material Description**
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
300*	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE – Reinforced Concrete Raft Slab **NOTE "*"**: OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements, however, is recommended to be a minimum of 300 mm to support the proposed rigid pavement structure.	

Table 9 - Recommended Pavement Structure – Access Lane, Fire Truck Lane, Ramp and Heavy Truck Parking Areas (Raft Slab)	
Thickness (mm)	**Material Description**
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
300*	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE – Reinforced Concrete Raft Slab **NOTE "*"**: OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements, however, is recommended to be a minimum of 300 mm to support the proposed rigid pavement structure.	

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.

For areas where silty clay is encountered at subgrade level and where overburden will be at the pavement structure subgrade, it is recommended that subdrains be installed during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Groundwater Suppression System

It is recommended that a groundwater suppression system be provided for the proposed structure since it will be founded below the current understood groundwater table. It is expected that insufficient room will be available for exterior backfill and the foundation walls will be cast as a blind-sided pour against a shoring system. It is recommended that the groundwater suppression system consist of the following:

❑ A waterproofing membrane should be placed against the shoring system between underside of the raft slab (recommended to extend the membrane a minimum of 600 mm horizontally below the raft slab) and a geodetic elevation of **66.0 m**.

❑ A composite drainage membrane (CCW MiraDRAIN 2000 or Delta-Teraxx or equivalent other reviewed and approved by Paterson) should be placed against the waterproofing membrane with the geotextile layer of the drainage board layer facing the waterproofing layer from an elevation of 66.0 m surface to the top of the raft. Above an elevation of 66.0 m, the foundation drainage board will be placed directly against the shoring system and/or overburden and will not be covered by the waterproofing membrane. Provisions should be carried for a second waterproofing membrane that will be advised by others to be placed between the foundation wall and HDPE-side of the drainage board layer for the height of the foundation.

❑ The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by Paterson field personnel.

❑ It is recommended that 150 mm diameter PVC sleeves at 6 m centers be cast in the foundation wall at the foundation wall/raft interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The sleeves should be connected to openings in the HDPE face of the drainage board layer and <u>should not cross the waterproofing membrane layer</u>.

- The perimeter drainage pipe and underfloor drainage system (detailed in subsequent paragraphs) should direct water to sump pit(s) within the lower basement area via an underfloor and interior drainage pipe system.

The top endlap of the foundation drainage board should be provided with a suitable termination bar against the foundation wall to mitigate the potential for water to perch between the drainage board and foundation wall.

Elevator Shaft and Additional Sub-Floor Structures Waterproofing

Elevator shafts located below the underslab drainage system should be provided full-depth positive-side waterproofing and provided with a PVC waterstop at the shaft wall and footing interface. Review of architectural design drawings should be completed by Paterson for the above-noted items once the building design has been finalized and prior to tender.

A positive-side (i.e., placed on exterior faces) waterproofing system should also be provided for any elevator shafts, pools, cisterns and other structures water-tight structures that will be located within the lowest basement level.

Interior Perimeter and Underfloor Drainage

An interior perimeter and underfloor drainage system will be required to control water infiltration below the lowest underground parking level slab and redirect water from the building's foundation drainage system to the buildings sump pit(s). The interior perimeter and underfloor drainage pipe should consist of a 100 to 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided with tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves. The spacing of the underfloor drainage system should be provided by Paterson once the foundation and column layout and sump system location(s) have been finalized and during the design phase of the project (i.e., prior to tender).

Review of Architectural and Waterproofing/Drainage System Designs

Since a groundwater suppression and underfloor drainage system designed by Paterson is being recommended to be implemented at the subject site, Paterson should review and advise on the architectural design of these features during the design phase and prior to tender.

Foundation Raft Slab Construction Joints

It is anticipated the raft slab will be poured in several pour segments. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab. Additional efforts, such as placing waterproofing membrane layers along the exterior side of cold joints in elevator shaft pours and other sections that require localized deepened pours, may be advised by Paterson during the design phase.

Foundation Backfilling

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 CCW MiraDRAIN 2000 or Delta-Teraxx, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection Against Frost Action

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

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

The foundations for the underground parking levels are expected to have sufficient frost protection due to the founding depth. However, it has been our experience that insufficient soil cover is typically provided to entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided for these areas.

6.3 Excavation Side Slopes

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

It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e., unsupported excavations).

Unsupported Excavations

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 Type 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

It is expected temporary shoring will be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. This is expected based on the proximity of the existing structures and roadways. 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 soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be included to the earth pressures described below. 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 using the parameters provided in Table 10.

Table 10 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
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 (γ), kN/m^3	20
Submerged Unit Weight (γ'), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

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

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

6.4 Pipe Bedding and Backfill

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

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular A. The bedding layer thickness should be increased to a minimum of 300 mm where the subgrade will consist of grey silty clay. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

It should generally be possible to re-use the moist (not wet) site-generated fill above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet site-generated fill, such as the grey silty clay, will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

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

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

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. It is recommended that hydraulic conductivity testing be complete during the design phase by Paterson to better estimate the volume of influx that may be handled during the construction phase.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

A temporary Ministry of 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 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, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

Groundwater Lowering and Adverse Effects on Neighboring Structures

Paterson anticipates the proposed structure will be founded below the groundwater table and has advised to incorporate a groundwater suppression system in the buildings waterproofing design. The groundwater suppression system will mitigate the potential for localized long-term dewatering of the clay deposit by the buildings foundation drainage system. Based on that, the proposed development will not result in long-term dewatering of the local groundwater table and/or subsequent adverse effects of neighboring structures.

6.6 Winter Construction

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

Where excavations are completed in proximity to existing structures which may be adversely affected due to the freezing conditions. 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 particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract documents to protect the walls of the excavations from freezing, if and where applicable.

In the event of construction during below zero temperatures, the founding medium should be protected from freezing temperatures by the installation of straw, propane heaters and/or glycol lines 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 foundation is protected with sufficient soil cover to prevent freezing at founding level.

It is recommended that Paterson review/advises on plans for protecting the bearing medium should the foundation excavation be planned to be undertaken during winter months.

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

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

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 slightly aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Considerations

It is understood the proposed building will include one level of underground parking and the structures will be founded at a minimum of 5 m below finished grade. Given the depth of foundations proposed for the structure, it is expected that the support of the foundations derives from soil located below the depth that dewatering by tree roots.

Therefore, foundation distress due to potential moisture depletion caused by trees is not expected to occur at the subject site. Since the proposed structure is not anticipated to be founded upon silty clay soils affected by the depth of root penetration, City approved trees within the subject site will not be subject to planting restrictions as based on the *City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines*) from a geotechnical perspective.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

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

- Review of the geotechnical aspects of the excavation contractor's shoring design, prior to construction, if applicable.

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

- Review and inspection of the installation of the foundation drainage system, including installation of underfloor drainage systems and waterproofing of elevator shafts.

- Observation of all bearing surfaces prior to the placement of concrete.

- Sampling and testing of the concrete and fill materials.

- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

- Observation of all subgrades prior to backfilling 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 that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

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 Azure Urban Developments Inc. 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
April 15, 2024
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Escandar Abdullah, B. Eng.

Drew Petahtegoose, P.Eng.

Report Distribution:

- ❑ Azure Urban Developments Inc. (email copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMITS TESTING RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev.- 254 Argyle Avenue
Ottawa, Ontario

EASTING: 368075.107 **NORTHING:** 5030391.95 **ELEVATION:** 69.30

DATUM: Geodetic

REMARKS:

BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** March 13, 2024

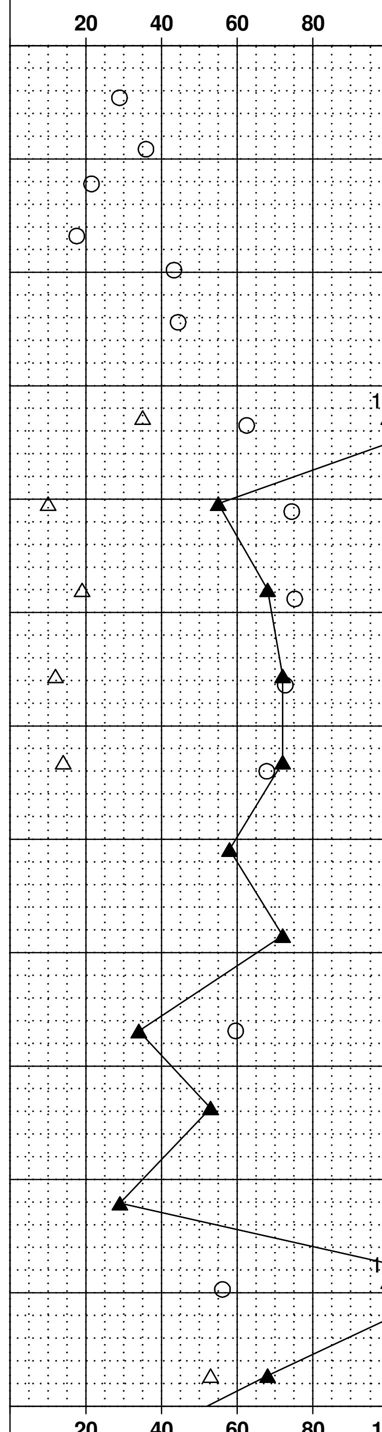
FILE NO. PG7026

HOLE NO. BH 1-24

SAMPLE DESCRIPTION	Depth to base of layer (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL, some clay, trace gravel	0.76	AU	1		
FILL: Brown silty clay, trace sand, gravel and topsoil	1.07	SS	2	25	8
FILL: Brown silty sand, trace clay, gravel and topsoil	1.83	SS	3	67	12
Very stiff to stiff, brown **SILTY CLAY**	3.73	SS	4	83	8
		SS	5	100	P
Stiff, grey **SILTY CLAY**		SS	6	100	P
		SS	7	100	P
		SS	8	100	P
		SS	9	100	P
- firm by 8.2m depth		SS	10	100	P
		SS	11	51	P

DEPTH (m)	ELEV. (m)
0	69.30
1	68.30
2	67.30
3	66.30
4	65.30
5	64.30
6	63.30
7	62.30
8	61.30
9	60.30
10	59.30
11	58.30
12	57.30

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



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

MONITORING WELL CONSTRUCTION

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev.- 254 Argyle Avenue
Ottawa, Ontario

EASTING: 368075.107 **NORTHING:** 5030391.95 **ELEVATION:** 69.30

DATUM: Geodetic

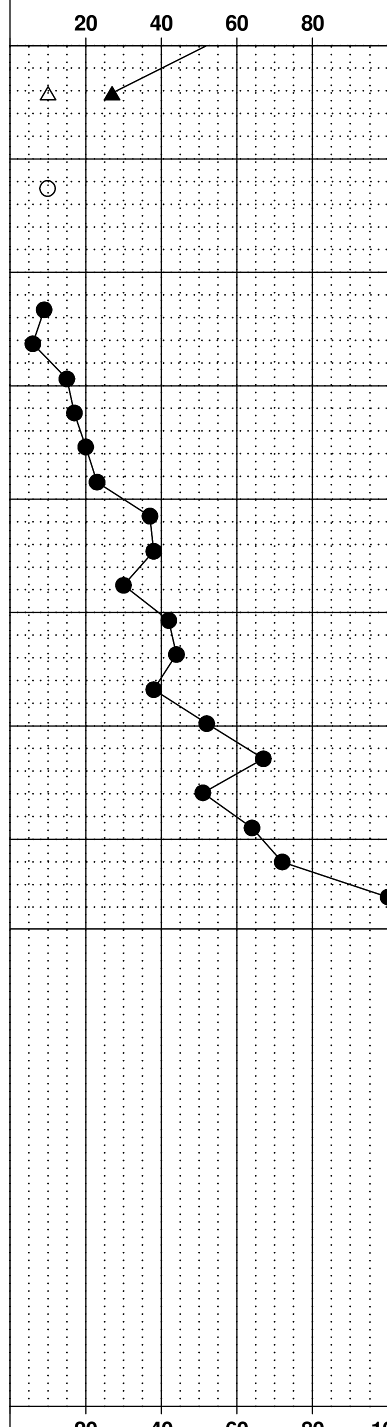
REMARKS:

BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** March 13, 2024

FILE NO. PG7026

HOLE NO. BH 1-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows / 0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	MONITORING WELL CONSTRUCTION
GROUND SURFACE						12	57.30		
12.80									
GLACIAL TILL: Compact to dense, grey silty clay with sand, gravel, cobbles and boulders		SS	12	100	+50	13	56.30		
13.72									
Dynamic Cone Penetration Test commenced at 13.72m depth. The cone was pushed to 14.33m depth.						14	55.30		
						15	54.30		
						16	53.30		
						17	52.30		
						18	51.30		
						19	50.30		
19.79									
End of Borehole Practical refusal to DCPT at 19.79m depth									



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev.- 254 Argyle Avenue
Ottawa, Ontario

EASTING: 368074.394 **NORTHING:** 5030392.805 **ELEVATION:** 69.42

DATUM: Geodetic

REMARKS:

BORINGS BY: CME 55 Low Clearance Power Auger **DATE:** March 13, 2024

FILE NO. PG7026

HOLE NO. BH 1A-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows / 0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	MONITORING WELL CONSTRUCTION
GROUND SURFACE						0	69.42		
TOPSOIL, some clay, trace gravel — 0.76									
FILL: Brown silty clay, trace sand, gravel and topsoil — 1.07						1	68.42		
FILL: Brown silty sand, trace clay, gravel and topsoil — 1.83						2	67.42		
Very stiff to stiff, brown **SILTY CLAY** — 3.73						3	66.42		
Stiff, grey **SILTY CLAY** — 4.27						4	65.42		
End of Borehole									
(GWL @ 2.95m - March 27, 2024)									

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

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev.- 254 Argyle Avenue
Ottawa, Ontario

EASTING: 368076.296 **NORTHING:** 5030384.839 **ELEVATION:** 69.55

DATUM: Geodetic

REMARKS:

BORINGS BY: Backhoe **DATE:** March 5, 2024

FILE NO. PG7026

HOLE NO. TP 1-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	69.55
TOPSOIL with organics		G	1				
0.40							
FILL: Brown silty sand with gravel, bricks, concrete and asphalt		G	2				
						1	68.55
		G	3				
1.70							
FILL: Brown silty sand with gravel		G	4				
1.90							
Very stiff, brown **SILTY CLAY**		G	5				
						2	67.55
2.40							
End of Test Pit							

(Test pit dry upon completion)

- Existing foundation wall appeared to consist of cast in place concrete.

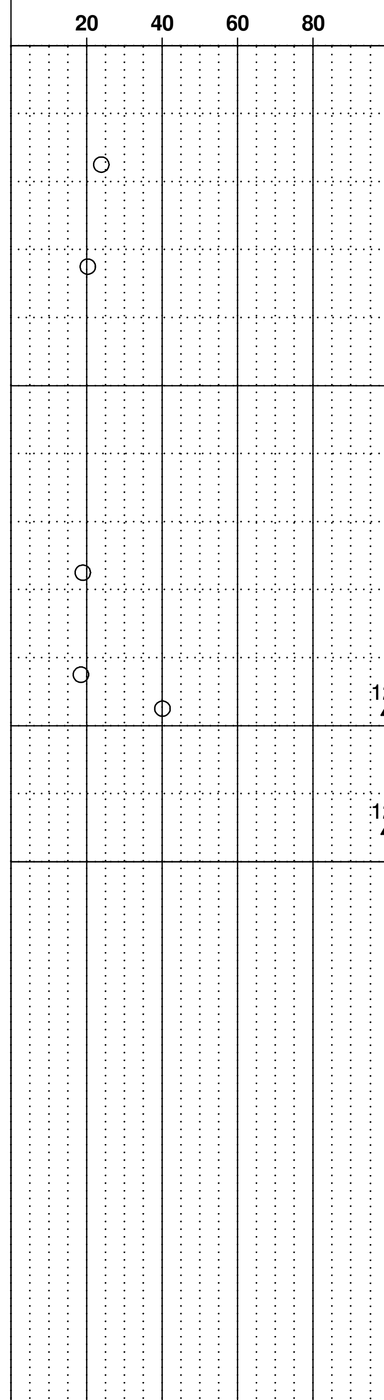
- Foundation wall observed at approximate depth of 1.4 mbgs (elevation of 68.15 m).

- Wall surface appeared to have damp proofing (black coat of tar like).

- No Drainage pipe was Observed.

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

PIEZOMETER CONSTRUCTION

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Dev.- 254 Argyle Avenue
Ottawa, Ontario

EASTING: 368068.223 **NORTHING:** 5030376.894 **ELEVATION:** 69.44

DATUM: Geodetic

REMARKS:

BORINGS BY: Backhoe **DATE:** March 5, 2024

FILE NO. PG7026

HOLE NO. TP 2-24

SAMPLE DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	69.44
TOPSOIL with organics (to 0.30)		G	1				
FILL: Brown silty sand with gravel, bricks, concrete and asphalt		G	2			1	68.44
(to 1.70)		G	3				
FILL: Brown silty sand with gravel (to 2.00)		G	4			2	67.44
Very stiff, brown **SILTY CLAY** (to 2.40)		G	5				

End of Test Pit

(Test pit dry upon completion)

- Existing foundation wall appeared to consist of cast in place concrete.

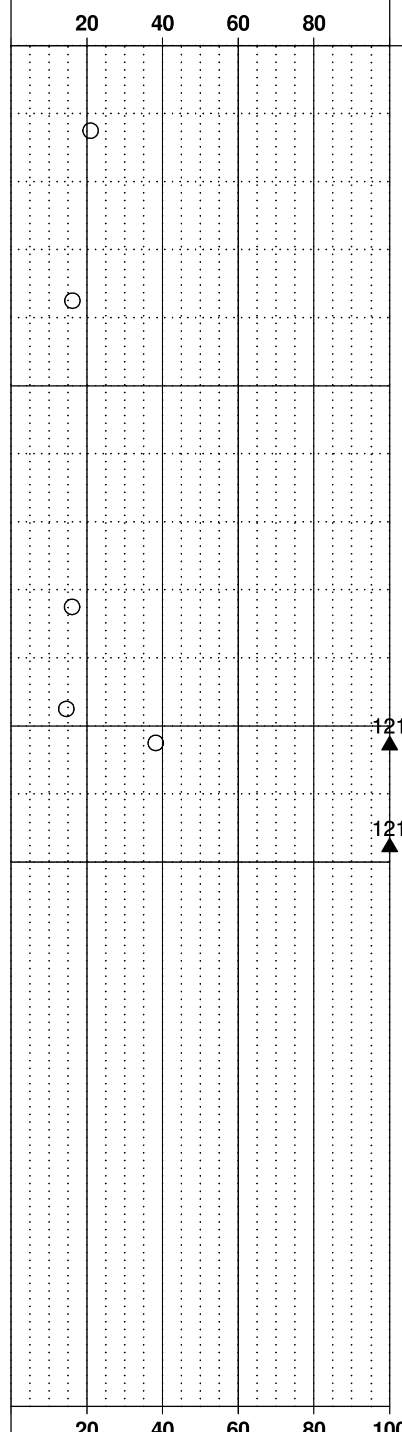
- Foundation wall observed at approximate depth of 1.4 mbgs (elevation of 68.04 m).

- Wall surface appeared to have damp proofing (black coat of tar like).

- No Drainage pipe was Observed.

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

PIEZOMETER CONSTRUCTION

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
254 Argyle Avenue
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located at the corner of Argyle Avenue and Bank Street. Geodetic elevation = 69.40m.

REMARKS

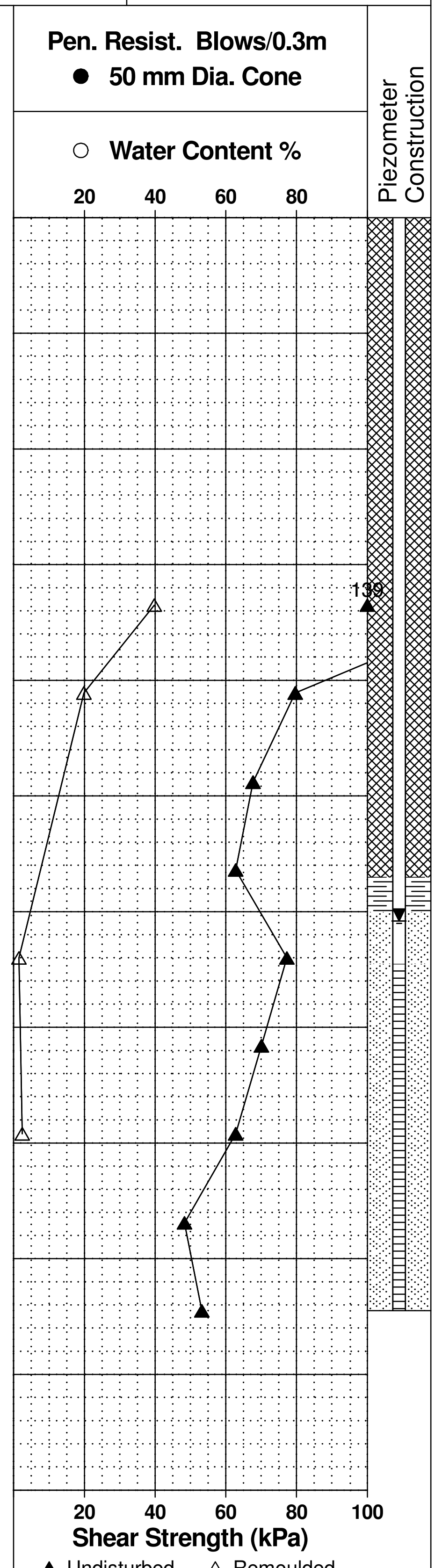
BORINGS BY CME 55 Power Auger

DATE 2019 November 25

FILE NO. PG5142

HOLE NO. BH 1

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	69.41
TOPSOIL 0.23	AU	1				
FILL: Dark brown silty clay, some sand 1.07	SS	2	62	4	1	68.41
FILL: Brown silty sand with clay, some gravel 1.52						
Very stiff to stiff, brown **SILTY CLAY**	SS	3	92	6	2	67.41
	SS	4	100	6	3	66.41
- grey by 3.8m depth	SS	5	100	P	4	65.41
					5	64.41
					6	63.41
					7	62.41
					8	61.41
9.45					9	60.41
Dynamic Cone Penetration Test commenced at 9.45m depth. Cone pushed to 14.9m depth.					10	59.41
					11	58.41



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
254 Argyle Avenue
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located at the corner of Argyle Avenue and Bank Street. Geodetic elevation = 69.40m.

REMARKS

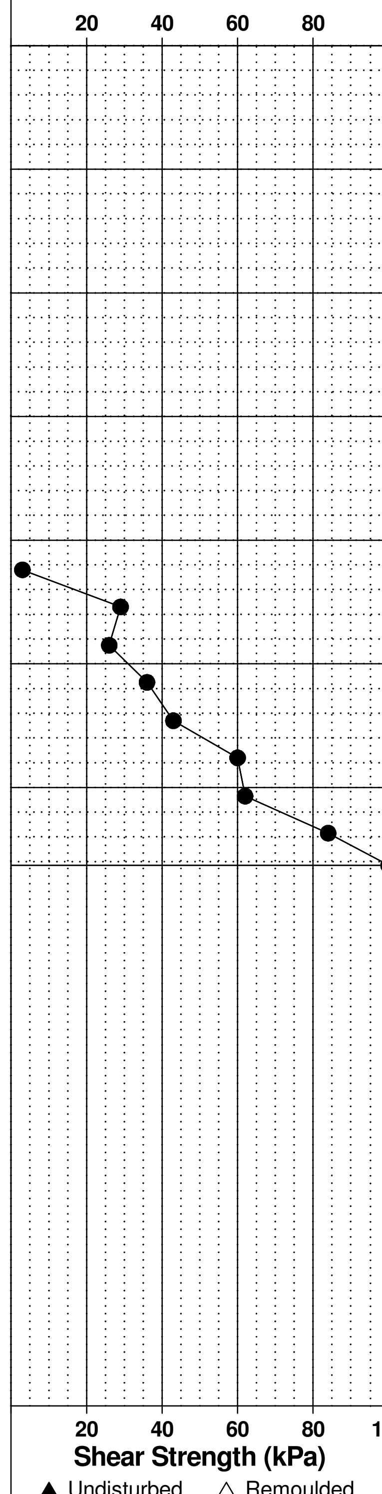
BORINGS BY CME 55 Power Auger

DATE 2019 November 25

FILE NO. PG5142

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE									
						11	58.41		
						12	57.41		
						13	56.41		
						14	55.41		
						15	54.41		
						16	53.41		
						17	52.41		
17.63									
End of Borehole Practical DCPT refusal at 17.63m depth. (GWL @ 6.08m - Nov. 29, 2019)									



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
254 Argyle Avenue
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin located at the corner of Argyle Avenue and Bank Street. Geodetic elevation = 69.40m.

REMARKS

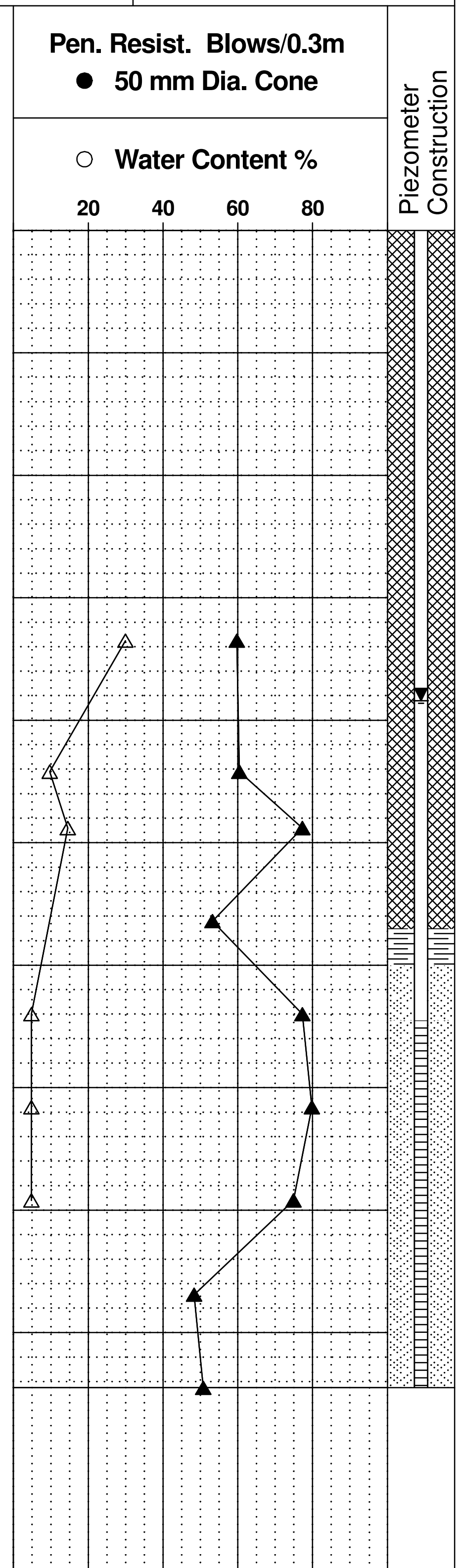
BORINGS BY CME 55 Power Auger

DATE 2019 November 25

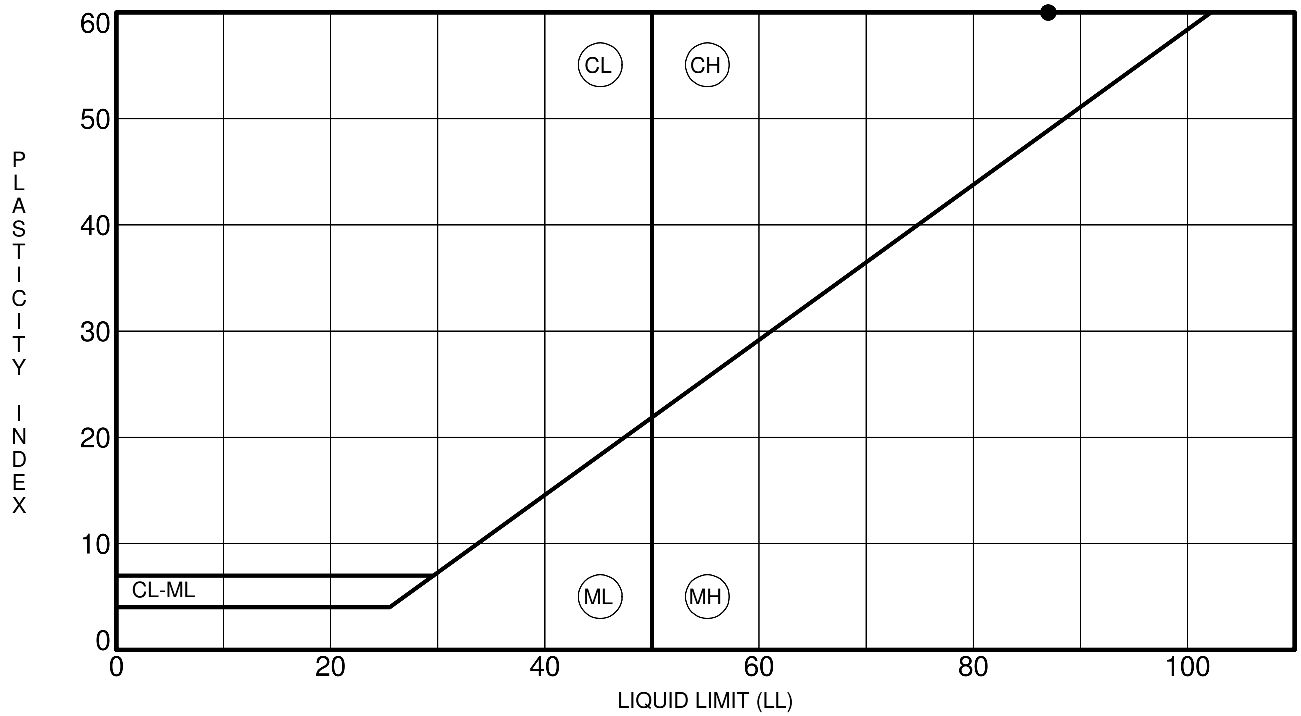
FILE NO. PG5142

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.47
FILL: Dark brown silty sand with clay and gravel (0.36)		AU	1				
FILL: Brown sand with gravel, some silt and clay (1.52)		SS	2	33	5	1	68.47
Stiff, brown **SILTY CLAY**		SS	3	88	9	2	67.47
		SS	4	0	4	3	66.47
- grey by 3.4m depth		SS	5	0	P	4	65.47
						5	64.47
						6	63.47
						7	62.47
						8	61.47
(9.45)						9	60.47
End of Borehole							
(GWL @ 3.84m - Nov. 29, 2019)							



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



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1-24**	**SS5**	**87**	**27**	**60**		**CH - Inorganic clays of high plasticity**

CLIENT	**Azure Urban**	FILE NO.	**PG7026**
PROJECT	**Geotechnical Investigation - Prop. High-Rise Dev.-**	DATE	**13 Mar 24**
	254 Argyle Avenue		

patersongroup **Consulting Engineers**

9 Auriga Drive, Ottawa, Ontario K2E 7T9

ATTERBERG LIMITS' RESULTS

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 = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = $D60 / D10$

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

CONSOLIDATION TEST

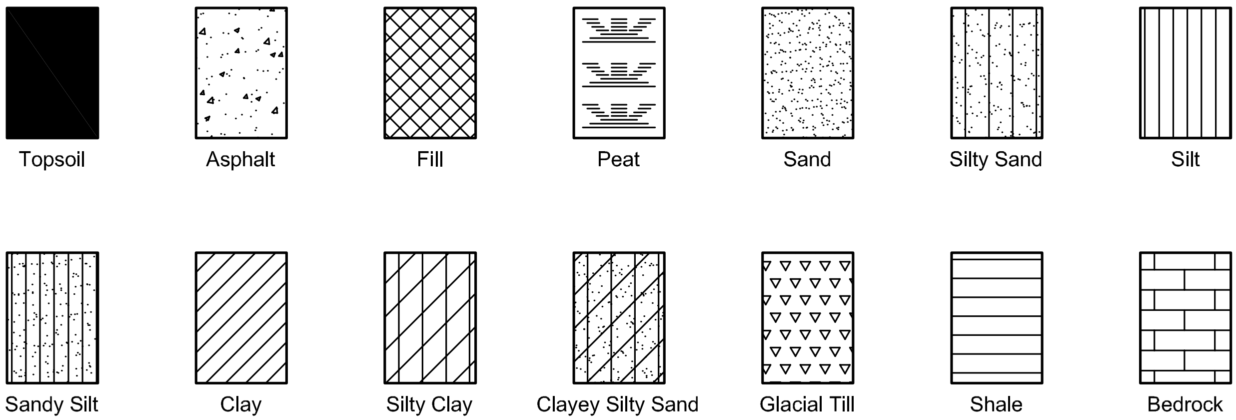
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

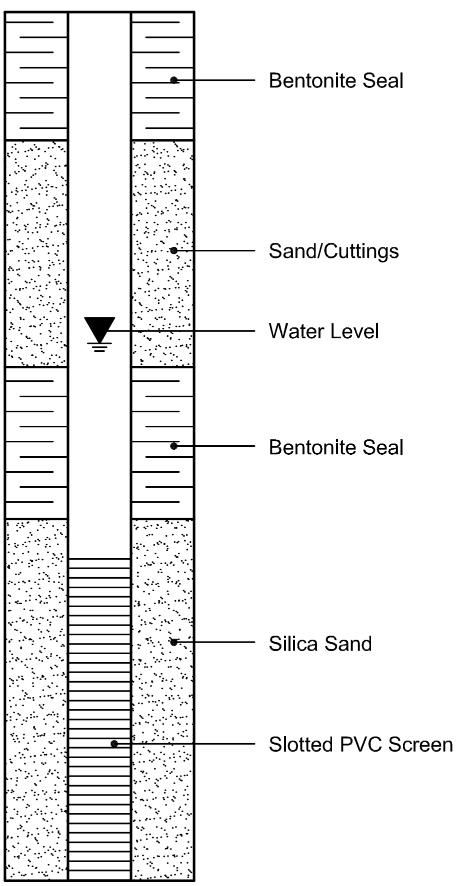
SYMBOLS AND TERMS (continued)

STRATA PLOT

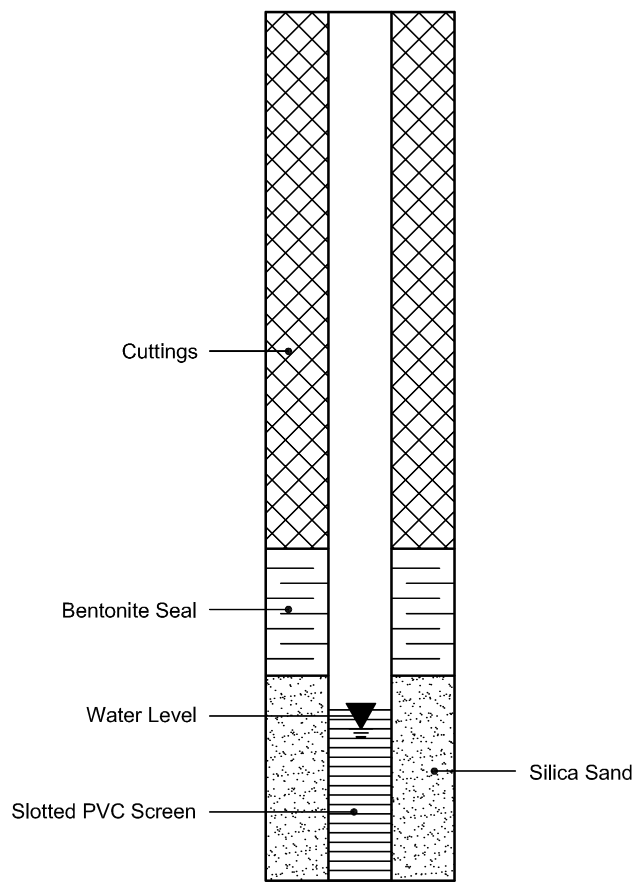


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

Order #: 2411420

Certificate of Analysis

Report Date: 20-Mar-2024

Client: Paterson Group Consulting Engineers (Ottawa)

Order Date: 14-Mar-2024

Client PO: 59668

Project Description: PG7026

	Client ID:	BH1-24-SS4	-	-	-	-
	Sample Date:	13-Mar-24 09:00	-	-	-	-
	Sample ID:	2411420-01	-	-	-	-
	Matrix:	Soil	-	-	-	-
	MDL/Units					
Physical Characteristics						
% Solids	0.1 % by Wt.	66.4	-	-	-	-
General Inorganics						
pH	0.05 pH Units	6.94	-	-	-	-
Resistivity	0.1 Ohm.m	49.4	-	-	-	-
Anions						
Chloride	10 ug/g	42	-	-	-	-
Sulphate	10 ug/g	51	-	-	-	-

APPENDIX 2

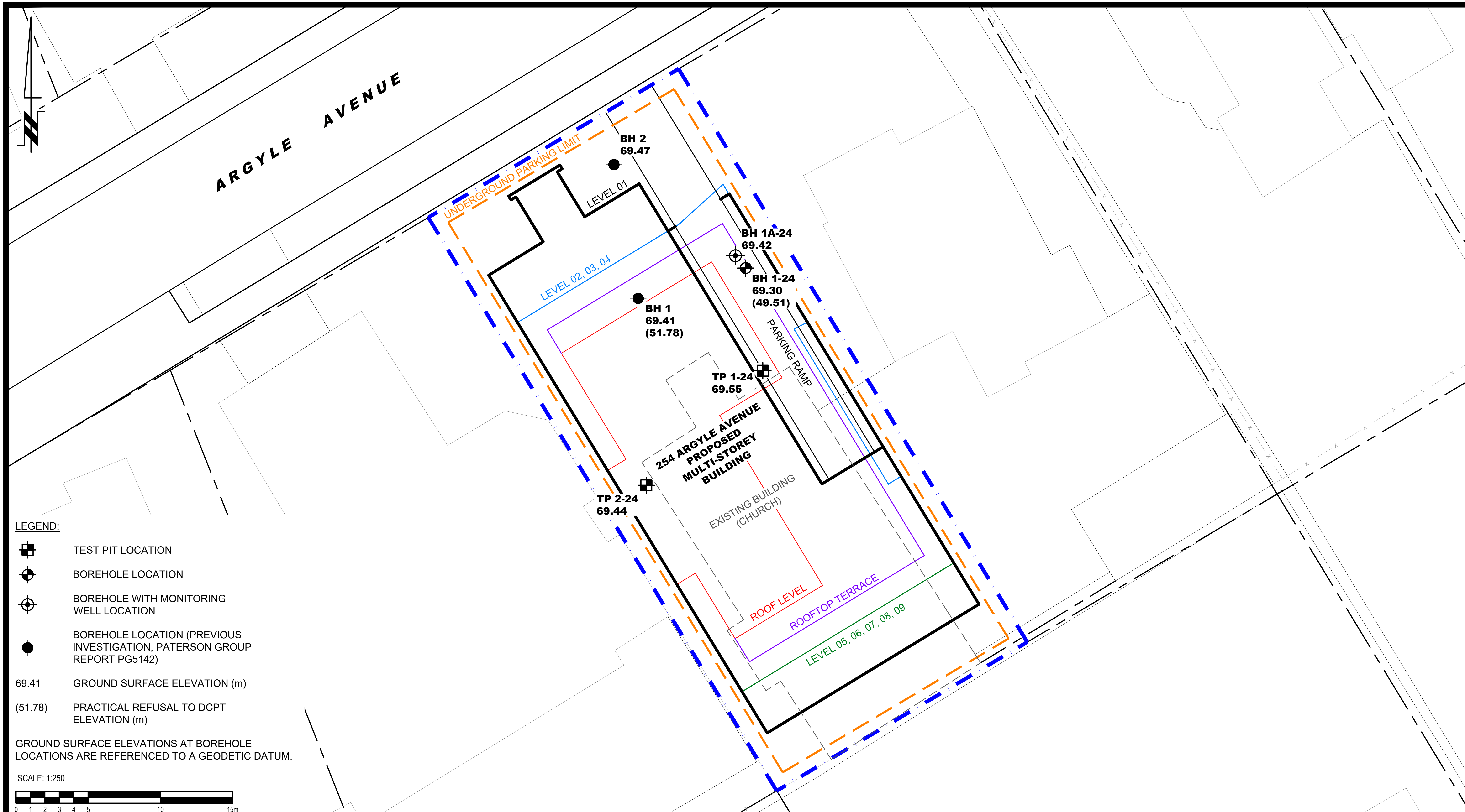
FIGURE 1 – KEY PLAN

DRAWING PG7026-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON GROUP



LEGEND:

- TEST PIT LOCATION
- BOREHOLE LOCATION
- BOREHOLE WITH MONITORING WELL LOCATION
- BOREHOLE LOCATION (PREVIOUS INVESTIGATION, PATERSON GROUP REPORT PG5142)

69.41 GROUND SURFACE ELEVATION (m)

(51.78) PRACTICAL REFUSAL TO DCPT ELEVATION (m)

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

SCALE: 1:250

0 1 2 3 4 5 10 15m

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

NO.	REVISIONS	DATE	INITIAL

AZURE URBAN DEVELOPMENTS INC.
GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE DEVELOPMENT
254 ARGYLE AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 03/2024
Drawn by: ZS	Report No.: PG7026-1
Checked by: EA	Dwg. No.: PG7026-1
Approved by: DP	Revision No.:

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