PATERSON GROUP

Geotechnical Investigation

Proposed Multi-Storey Building

2380 Tenth Line Road
Ottawa, Ontario

Prepared for Claridge Homes

Report PG7518-1 dated May 23, 2025

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes to conduct a geotechnical investigation for the proposed development to be located at 2380 Tenth Line Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

- ❑ Determine the subsoil and groundwater conditions at this site by means of boreholes, and to

- ❑ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Development

Although drawings were not available during the preparation of this report, it is anticipated that the proposed development will consist of a multi-storey building with 1 underground parking level. Associated at-grade access lanes, parking areas, and landscaped areas are also anticipated immediately around the proposed buildings. It is expected that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The current geotechnical investigation was carried out on May 2, 2025 and consisted of a total of 3 boreholes advanced to maximum depth of 10.5 m below the existing grade. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the boreholes are shown on Drawing PG7518-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

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

A Standard Penetration Test (SPT) was conducted at each borehole 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.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at borehole BH 2-25. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the 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.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

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

Flexible polyethylene standpipes were installed in all boreholes to measure the stabilized groundwater levels subsequent to the completion of the sampling program.

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

3.2 Field Survey

The borehole locations, and the ground surface elevation at each borehole location, were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The locations of the boreholes, and ground surface elevation at each borehole location, are presented on Drawing PG7518 -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 2 Atterberg limits tests, 1 shrinkage limit test, and 1 grain size distribution test were completed on selected soil samples obtained from the current geotechnical investigation.

All samples from the current investigation will be stored in the laboratory for 1 month after this report is completed. They will then be discarded unless we are otherwise directed.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

Currently, the subject site is mostly vacant and mostly grass-covered, although some construction materials are currently being stored in the north-central portion of the site. The site is bordered by a residential property to the north, Décoeur Drive to the south, Tenth Line Road to the east, and David Lewis Private to the west.

The ground surface across the site is relatively level at approximate geodetic elevation 88 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of an approximate 0.1 to 0.7 m thickness of fill underlain by a deep silty clay deposit.

The fill generally consists of brown silty clay to silty sand with varying amounts of gravel, crushed stone and organics.

Underlying the fill, a very stiff to stiff, brown silty clay was encountered, becoming firm to soft below an approximate depths of 2.6 to 3 m.

Practical refusal to the DCPT was encountered at borehole BH 2-25 at a depth of about 22.50 mm below the existing ground surface.

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

Bedrock

Based on available geological mapping, the bedrock in the subject area consists of interbedded shale and limestone bedrock of the Lindsay Formation with an overburden drift thickness ranging between 25 and 50 m.

Laboratory Testing

Atterberg limits testing, as well as associated moisture content testing, were completed on the recovered silty clay samples at selected locations throughout the subject site. The results of the Atterberg limits tests are presented in Table 1 and on the Atterberg Limits' Results sheet in Appendix 1.

Table 1 – Summary of Atterberg Limits Results

Test Hole	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Classification
BH 1-25	3.0 - 3.7	70	34	36	MH
BH 2-25	3.8 - 4.42	68	33	35	MH

Notes: CH: Inorganic Clay of High Plasticity; MH: Inorganic Silts of High Plasticity

Grain Size Distribution and Hydrometer Testing

Grain size distribution (sieve analysis) was also completed on 1 selected soil sample. The results of the grain size analysis are summarized in Table 2, and presented on the Grain-Size Distribution Testing Results sheets in Appendix 1.

Table 2 – Summary of Grain Size Distribution Analysis

Test Hole	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 1-25	3.8 - 4.42	0	0.2	39.3	60.5

Shrinkage Test

The results of the shrinkage limit test indicate a shrinkage limit of 35.51% and a shrinkage ratio of 1.762.

4.3 Groundwater

The observed groundwater levels are summarized in Table 3 on next page, and are also provided on the Soil Profile and Test Data sheets in Appendix 1.

Table 3 - Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Recording Date
		Groundwater Depth (m)	Groundwater Elevation (m)	
BH 1-25*	88.23	5.51	82.72	May 9, 2025
BH 2-25*	88.20	1.58	86.62	
BH 3-25*	87.86	1.80	86.06	

Note: The ground surface elevations from the current investigation are referenced to a geodetic datum.
* Borehole instrumented with piezometer.

Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected to be at a depth of approximately 2.5 to 3.5 m below the existing ground surface.

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

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. Foundation support for the proposed building is recommended to consist of:

- Conventional spread footings bearing on the silty clay which has been adequately improved with a ground improvement technique such as Controlled Modulus Columns (CMCs)

- Deep foundations consisting of end bearing piles extending to the bedrock

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 section 5.3.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, building debris, and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. 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 building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery 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 proposed building areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD). Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. In landscaped areas, 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 used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD.

Compacted Granular Fill Working Platform

The use of heavy equipment will be required to install the piles (i.e. pile driving crane) or CMCs (drill rig). 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.

A typical working platform could consist of 1 m of OPSS Granular B, Type II crushed stone which is placed and compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in lifts not exceeding 300 mm in thickness.

Once the piles or CMCs have been installed, the working platform can be regraded, 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 recompacted to act as the substrate for further fill placement for the basement slab.

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 into the construction operations to maintain, as much as possible, a cooperative environment with the residents.

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

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). It should be noted that these guidelines are for today’s construction standards.

Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

5.3 Foundation Design

Ground Improvement - Controlled Modulus Columns (CMCs)

As noted above, foundation support for the proposed building is recommended to consist of conventional spread footings bearing on undisturbed silty clay which has been adequately improved with a ground improvement technique such as Controlled Modulus Columns, or approved equivalent.

Controlled Modulus Columns (CMCs) involve advancing a reverse-flight auger to the target bearing stratum, which for this site consists of the bedrock, and which displaces and densifies the surrounding soils during the advancement process. A cement-based grout is then discharged under pressure at the base of the reverse-flight auger as it is retracted, creating a concrete column.

The CMCs are installed in a grid pattern to create a stiffened matrix of the concrete columns and densified in-situ soils for the support of conventional foundations. CMCs can be installed to depths of up to approximately 20 m.

Following installation of the CMCs, a load transfer platform, consisting of a specified thickness of compacted granular fill, is placed over the top of the CMCs. Ground improvement programs are completed by design-build contractors, many of which have proprietary systems. A ground improvement contractor specializing in these programs should be contacted to determine which system is most appropriate for the proposed development, and for more information regarding cost and design details.

However, for preliminary design, footings bearing on an undisturbed silty clay bearing surface which has been adequately improved using CMCs, or an equivalent ground improvement method which has been approved by Paterson, 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.** However, the final bearing resistance values which will be applicable will be provided by the specialized ground improvement contractor.

The ground improvement design shall be presented by the ground improvement contractor to Paterson as a technical submittal for review during the design phase of the project.

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 improved silty clay bearing surface when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as that of the bearing medium.

End Bearing Pile Foundation

As an alternative to footing supported on CMCs, concrete-filled steel pipe piles could be utilized for foundation support of the proposed building. Applicable pile resistance values at ultimate limit states (ULS) are given in Table 4 below. Note that these are all geotechnical axial resistance values in compression.

Table 4 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance	Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		Factored at ULS (kN)		
245	9	1,090	10	50
245	11	1,260	11	50
245	13	1,500	12	50

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. Re-striking of all piles at least once will also be required after 48 hours have elapsed since initial driving.

The minimum centre-to-centre pile spacing is 2.5 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.

If there are proposed grade raises at the site, downdrag loads should be considered on the piles, this can be discussed further if piles are selected as the foundation support option.

Lateral and uplift capacities of the piles can also be provided if piles are selected as the foundation support option for the proposed building.

Permissible Grade Raise Restrictions

A permissible grade raise restriction of **0.8 m** is recommended for the subject site. If greater 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 seismic site designation can be taken as **Class X_E** for foundations constructed at the subject site. If a higher seismic site class is desired(**Class X_D**) for the proposed building, a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed building, as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2024.

The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the 2024 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill within the footprint of the proposed building, the undisturbed silty clay will be considered an acceptable subgrade upon which to commence backfilling for basement slab construction. It is anticipated that the basement area for the proposed building will be mostly parking and the recommended pavement structure noted in section 5.8 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

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

For any below-grade space, an underslab drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided under the lowest level floor slab. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the 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.

Two distinct conditions, static and seismic, should be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

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 this site is 0.423g for Site Class X_E according to the OBC 2024. Note that the vertical seismic coefficient is assumed to be zero.

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

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

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

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

5.7 Pavement Design

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 below.

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
SUBGRADE – Compact to dense glacial till, or OPSS Granular B Type I or II material material placed over bedrock.	

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

The flexible pavement structures presented in Tables 6 and 7 should be used for at grade access lanes and heavy loading parking areas.

Table 6 - Recommended Pavement Structure - Access Lanes

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 - Compact to dense glacial till, or OPSS Granular B Type I or II material material placed over bedrock.	

Table 7 - Recommended Pavement Structure – Car Only Parking Areas

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

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

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory 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.

Due to the impervious nature of the silty clay deposit, where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction. The subdrain inverts should be approximately 300 mm below subgrade level and run longitudinal along the curb lines. 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

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed building. The system should consist of a 100 mm diameter perforated and corrugated PVC pipe, surrounded on all sides by 100 mm of 19 mm clear crushed stone, which is placed at the foundation level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Underslab Drainage

Underslab drainage is recommended to control water infiltration for the basement area. For preliminary design purposes, we recommend that 100 mm diameter perforated PVC pipes be placed at approximately 6 m spacing. The spacing of the underslab drainage system should be confirmed at the completion time of the excavation when water infiltration can be better assessed.

Foundation Backfill

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

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover in conjunction with adequate foundation insulation, should be provided.

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

Unheated structures, such as the access ramp wall foundation, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover in conjunction with foundation insulation, should be provided for the ramp wall.

6.3 Excavation Side Slopes

Temporary Side Slopes

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

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. Excavations below the groundwater level should be cut back at a maximum slope of 1.5H:1V. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

Temporary shoring

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

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

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation event will not negatively impact the temporary shoring system or soils supported by the system. Any changes to the approved temporary shoring system design should be reported immediately to the owner's structural designer prior to implementation.

The temporary shoring system may consist of a soldier pipe and lagging system which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below.

The earth pressure acting on the shoring system may be calculated using the parameters in Table 8 below:

Table 8 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (Ka)	0.33
Passive Earth Pressure Coefficient (Kp)	3
At-Rest Earth Pressure Coefficient (Ko)	0.5
Unit Weight, kN/m3	21
Submerged Unit Weight, kN/m3	13

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

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. 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 & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

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

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

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.5 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

Clay Seals

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

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

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

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

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

Impacts on Neighbouring Structures

Based on observations, the groundwater level is anticipated at a 2.5 to 3.5 m depth. Localized and minor groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. The extent of any significant groundwater lowering should occur within a limited range of the subject site due to the minimal temporary groundwater lowering.

No issues are expected, with respect to groundwater lowering, that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

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

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

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

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 very aggressive corrosive environment.

6.8 Landscaping Considerations

Paterson completed a soils review of the site to determine the applicable tree planting setbacks, in accordance with the City of Ottawa's Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines). Atterberg limits testing was completed for selected silty clay samples. Sieve analysis testing was also completed on selected soil samples. The results of the testing are presented in Tables 1 and 2 in Section 4.2 and also in Appendix 1.

Based on the results of our review, the plasticity index of the silty clay deposit at the subject site does not exceed 40%. Therefore, the following tree planting setbacks are recommended for the silty clay deposit. Large trees (mature height over 14 m) can be planted within the silty clay areas provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g., in a park or other green space). Tree planting setback limits may be reduced to **4.5 m** for small (mature height up to 7.5 m) and medium size trees (mature tree height 7.5 to 14 m), provided that the conditions noted below are met.

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

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

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

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

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

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows, and some maples (i.e., Manitoba Maples) and, as such, they should not be considered in the landscaping design.

7.0 Recommendations

It is a requirement for the foundation data provided herein to be applicable that the following material testing, and observation program be performed by the geotechnical consultant.

- Reviewing of the grading and servicing plan, from a geotechnical perspective.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

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

8.0 Statement of Limitations

The recommendations provided herein are in accordance with 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 Claridge Homes or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
May 23, 2025
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Deepak K Rajendran, E.I.T.

Scott S. Dennis, P.Eng.

Report Distribution:

- ❑ Claridge Homes (Digital copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMITS RESULTS

GRAIN SIZE ANALYSIS RESULTS

SHRINKAGE LIMIT RESULTS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2380 Tenth Line Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 384504.24 **NORTHING:** 5034752.82 **ELEVATION:** 88.23

PROJECT: Proposed Multi-Storey Building

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: May 1, 2025

FILE NO. : PG7518

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION — GROUND SURFACE

- **FILL:** Gravel and crushed stone — 0.05m [88.18m]
- **FILL:** Compact, brown silty sand, with gravel, trace clay — 0.69m [87.54m]
- Very stiff to stiff, brown **SILTY CLAY**
  - Firm by 2.2 m depth
  - 3.00m [85.23m]
- Stiff to soft, grey **SILTY CLAY**
  - Stiff by 5.3 m depth
  - Soft by 6.1 m depth
  - Firm by 6.8 m depth
  - 10.52m [77.71m]

End of Borehole

(GWL at 5.51 m depth - May 9, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	REMOULDED SHEAR STRENGTH (kPa)	UNDRAINED SHEAR STRENGTH (kPa)
AU 1			3		
SS 2	42	2-2-4-7 6	38		
SS 3	100	2-2-3-3 5	47		249
SS 4	100	P	51	20	50
SS 5	100	P		10	29
SS 6	100	P		10	34
SS 7	67	P	56	12	29
SS 8	100	P	54	19	53
SS 9	58	P	81		24
SS 10	100	P	64		39
SS 11	75	P	81		39
SS 12	100	P	86		39
SS 13		P	79		39
SS 14	100	P	66		43

■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE)

△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa)

PL (%) WATER CONTENT (%) LL (%)

PIEZOMETER CONSTRUCTION — 5.51 m 2025-05-09

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2380 Tenth Line Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 384447.38 **NORTHING:** 5034743.07 **ELEVATION:** 88.20

PROJECT: Proposed Multi-Storey Building

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: May 1, 2025

FILE NO. : PG7518

HOLE NO. : BH 2-25

SAMPLE DESCRIPTION

GROUND SURFACE

Brown **SILTY CLAY**, trace organics 0.08m [88.12m]

Very stiff, brown **SILTY CLAY**

- Firm by 2.2 m depth

2.64m [85.56m]

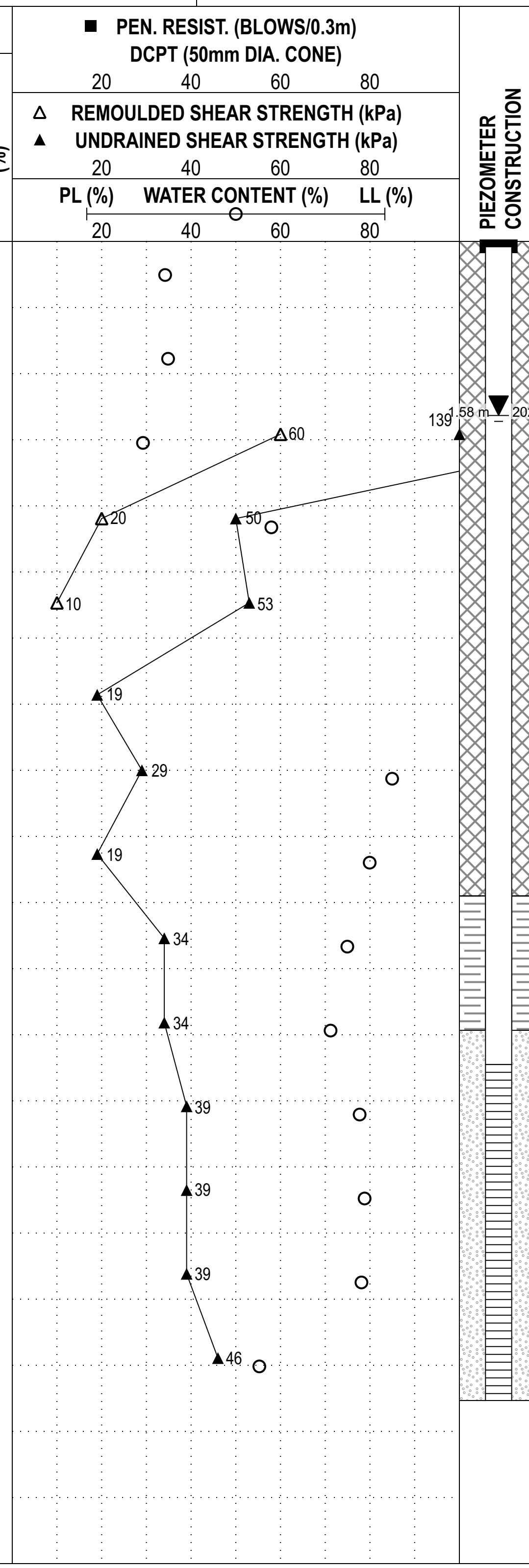
Firm to soft, grey **SILTY CLAY**

10.52m [77.68m]

Dynamic cone penetration test commenced at 10.52 m depth

SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			34
SS 2	58	1-3-6-8 9	35
SS 3	50	P	29
SS 4	100	P	58
SS 5	75	P	
SS 6	100	P	
SS 7	50	P	85
SS 8	100	P	80
SS 9	83	P	75
SS 10	100	P	71
SS 11	92	P	78
SS 12	100	P	79
SS 13	100	P	78
SS 14	100	P	55

STRATA PLOT | DEPTH (m) 1–12 | ELEVATION (m) 88–77



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2380 Tenth Line Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 384447.38 **NORTHING:** 5034743.07 **ELEVATION:** 88.20

PROJECT: Proposed Multi-Storey Building

ADVANCED BY: CME-55 Low Clearance Drill

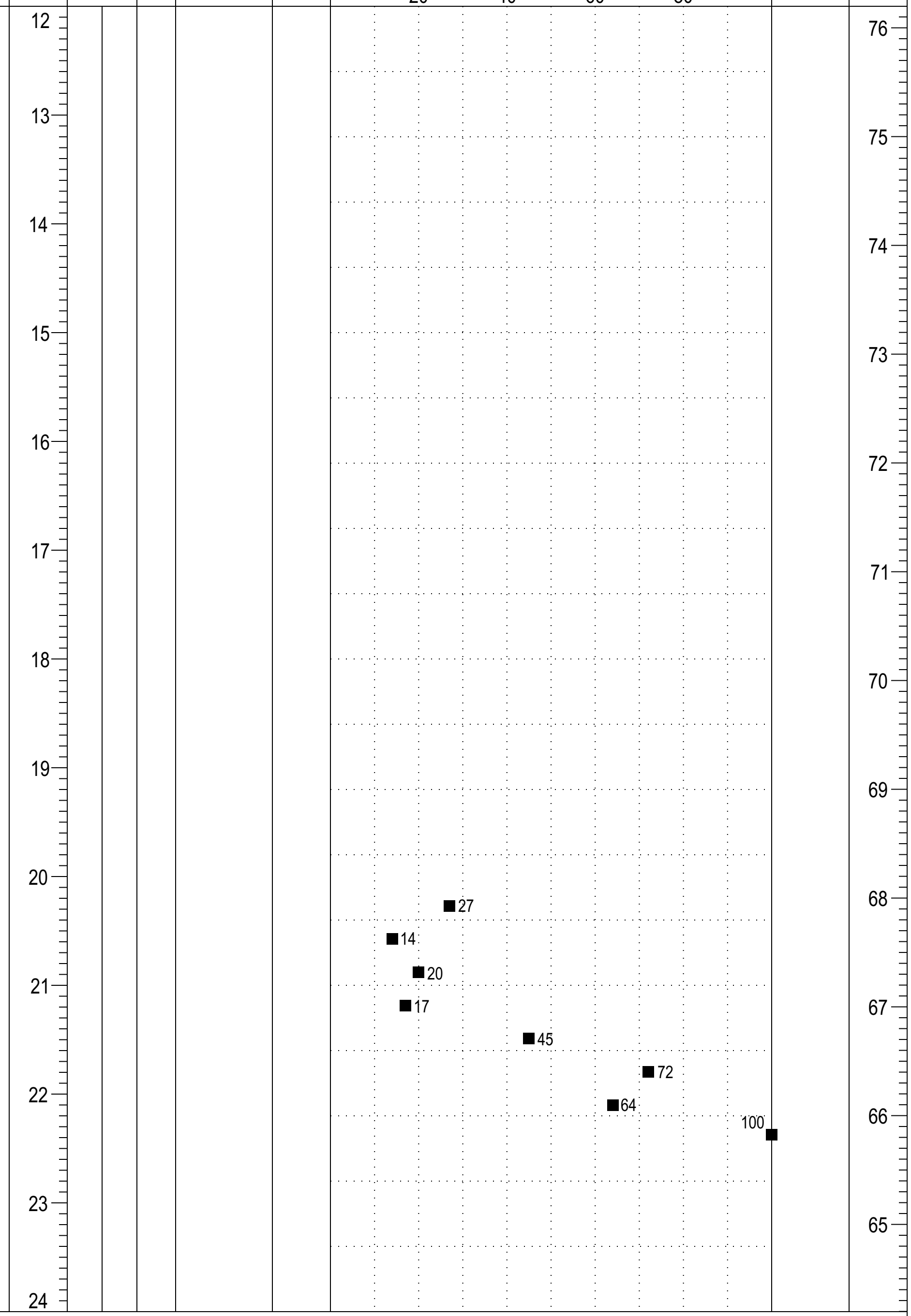
REMARKS:

DATE: May 1, 2025

FILE NO. : PG7518

HOLE NO. : BH 2-25

SAMPLE DESCRIPTION	STRATA PLOT	DEPTH (m)	SAMPLE: TYPE AND NO.	SAMPLE: RECOVERY (%)	SAMPLE: N OR RQD	SAMPLE: WATER CONTENT (%)	■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80; △ REMOULDED SHEAR STRENGTH (kPa) ▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80; PL (%) WATER CONTENT (%) LL (%) 20 40 60 80	PIEZOMETER CONSTRUCTION	ELEVATION (m)
Dynamic cone penetration test									
22.50m [65.70m]									
End of Borehole DCPT pushed from 10.52 m to 20.12 m depth Practical refusal to DCPT at 22.50 m depth (GWL at 1.58 m depth - May 9, 2025)									



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2380 Tenth Line Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 384443.03 **NORTHING:** 5034714.19 **ELEVATION:** 87.86

PROJECT: Proposed Multi-Storey Building

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: May 2, 2025

FILE NO. : PG7518

HOLE NO. : BH 3-25

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Brown silty clay, with gravel, some crushed stone — 0.46m [87.40m]

Very stiff to stiff, brown **SILTY CLAY** — 2.97m [84.89m]

Firm to soft, grey **SILTY CLAY**

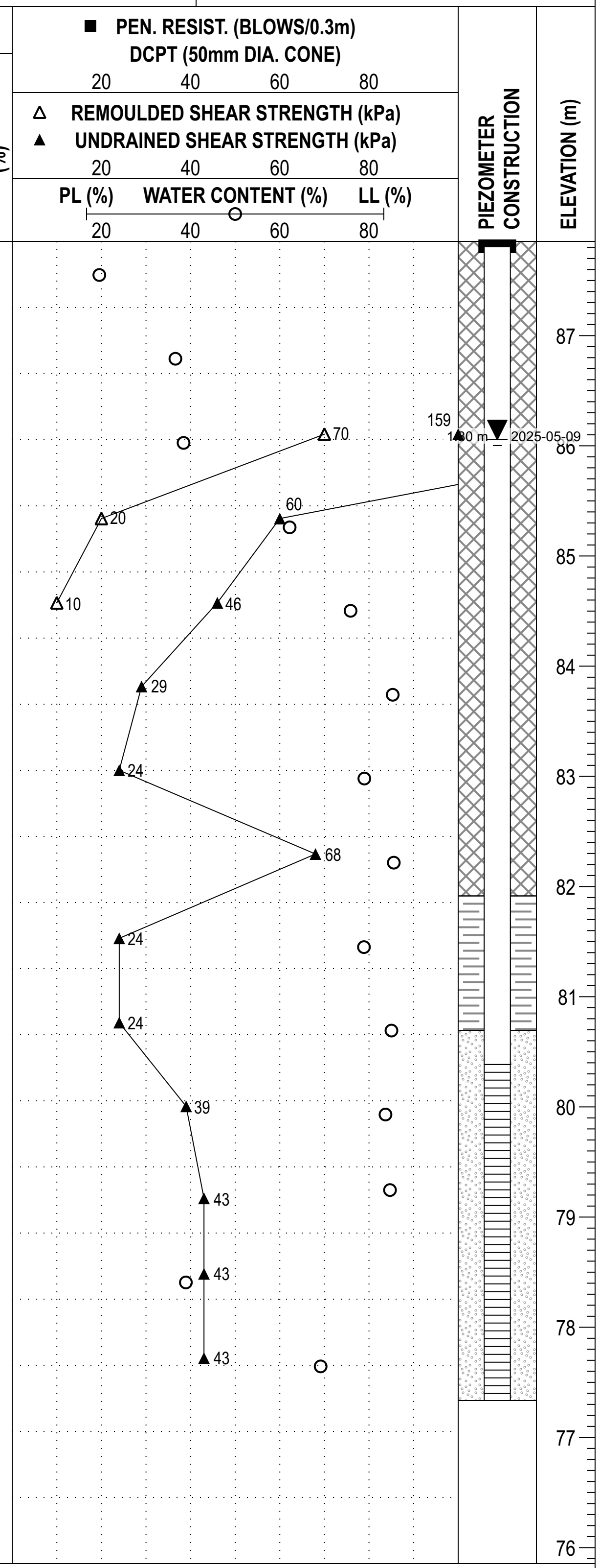
- Firm by 7.5 m depth

— 10.52m [77.34m]

End of Borehole

(GWL at 1.80 m depth - May 9, 2025)

SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			20
SS 2	50	1-0-3-3 3	37
SS 3	58	P	38
SS 4	100	P	62
SS 5	100	P	76
SS 6	100	P	85
SS 7	100	P	79
SS 8	100	P	86
SS 9	42	P	79
SS 10	100	P	85
SS 11	100	P	84
SS 12		P	85
SS 13	100	P	39
SS 14	100	P	69



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their "sensitivity". The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

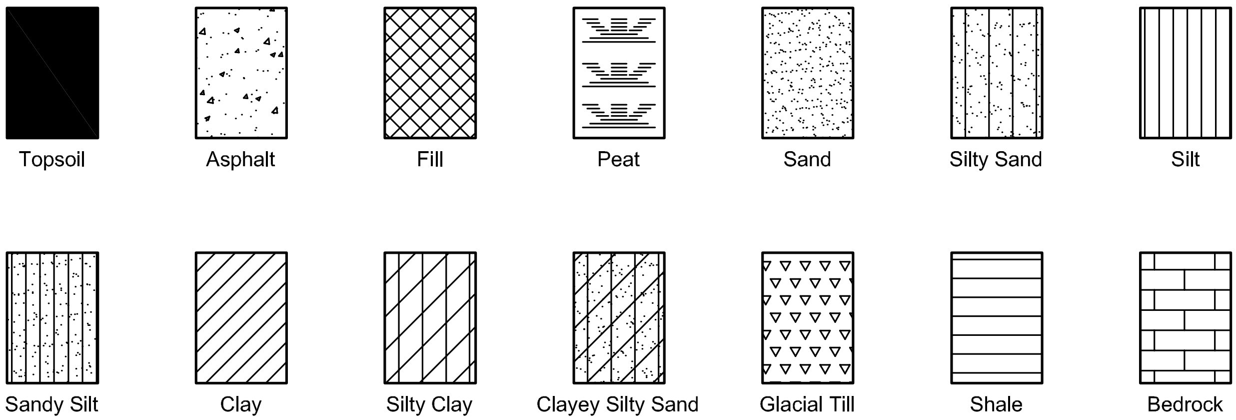
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

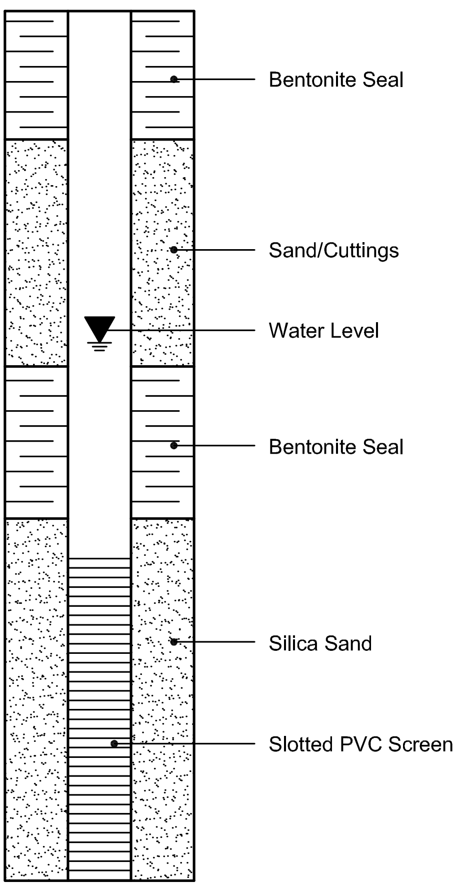
SYMBOLS AND TERMS (continued)

STRATA PLOT

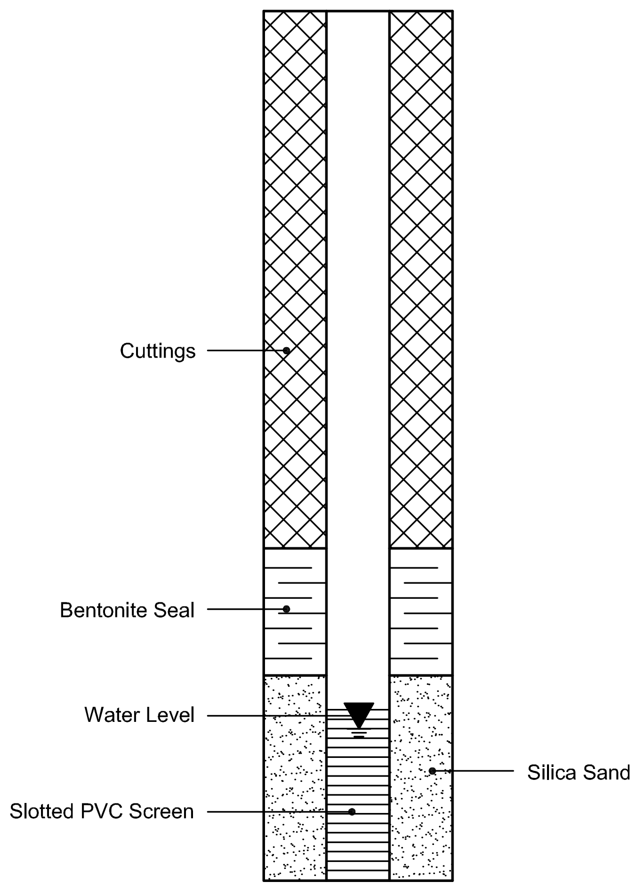


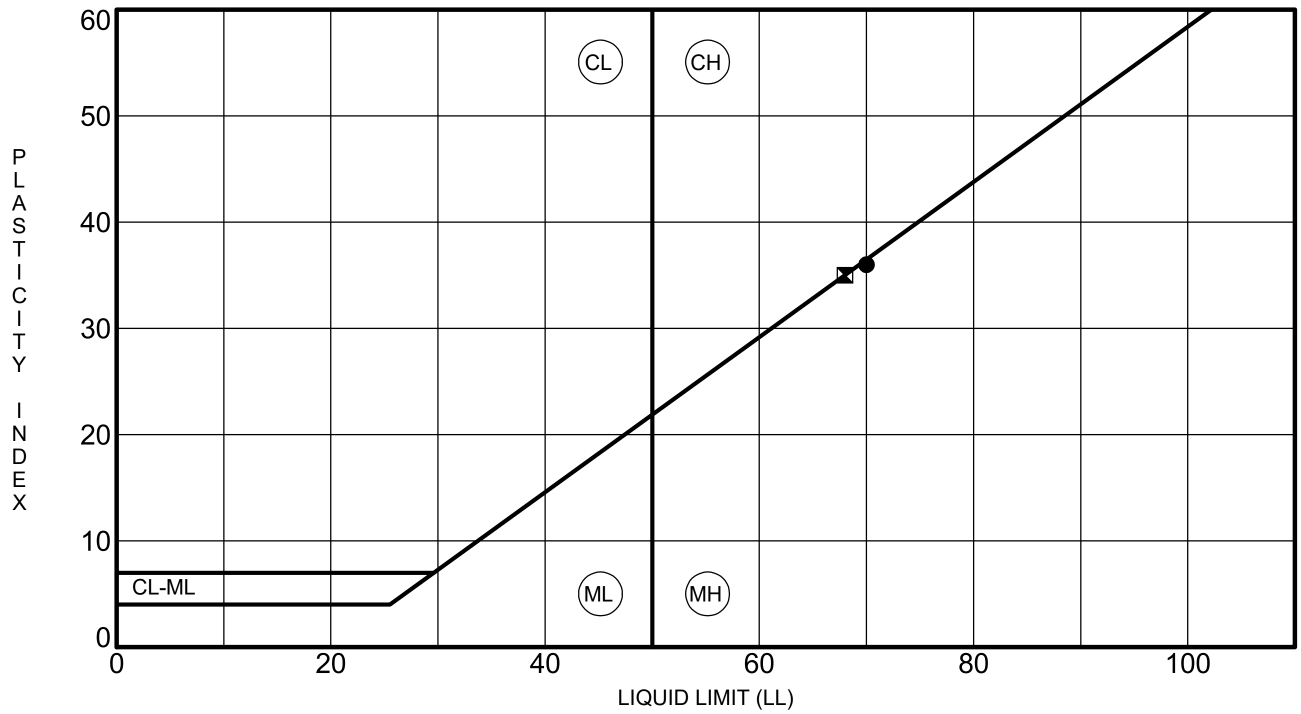
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1-25**	**SS5**	**70**	**34**	**36**		**MH - Inorganic silts of high plasticity**
⧗	**BH 2-25**	**SS6**	**68**	**33**	**35**		**MH - Inorganic silts of high plasticity**

CLIENT **Claridge Homes**

FILE NO. **PG7518**

PROJECT **Geotechnical Investigation - Proposed Multi-Storey Building - 2380 Tenth Line Road, Ottawa, Ontario**

DATE **14 May 25**

PATERSON GROUP

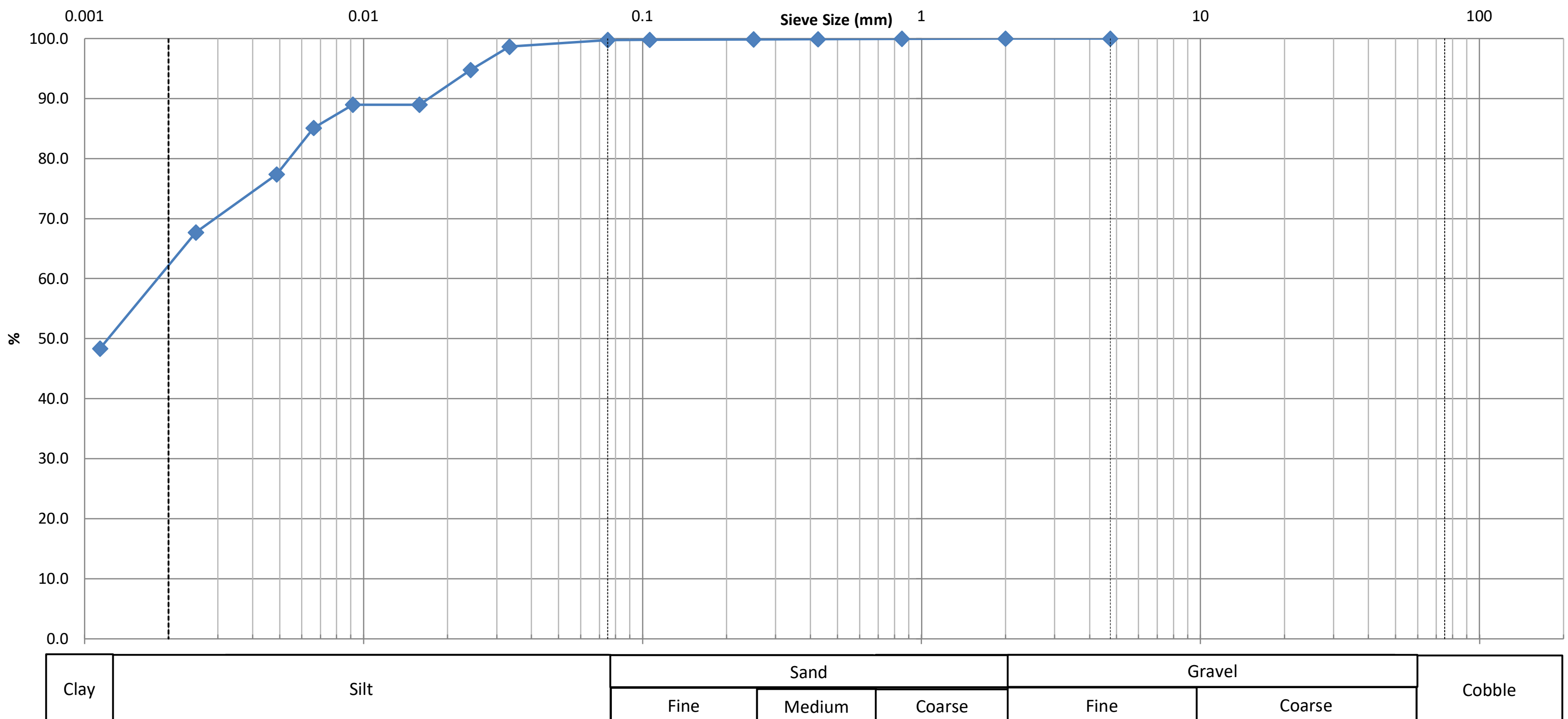
9 Auriga Drive
Ottawa, Ontario
K2E 7T9
TEL: (613) 226-7381

ATTERBERG LIMITS' RESULTS

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	Claridge Homes	DEPTH:	BH1-25 SS6	FILE NO:	PG7518
CONTRACT NO.:		BH OR TP No.:	-	LAB NO:	59377
PROJECT:	2380 Tenth Line Road, Ottawa, Ontario			DATE RECEIVED:	2-May-25
				DATE TESTED:	5-May-25
DATE SAMPLED:	2-May-25			DATE REPORTED:	14-May-25
SAMPLED BY:	-			TESTED BY:	D.K



Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
						59.1%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					0.0	0.2		39.3		60.5	

Comments:

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.

patersongroup
consulting engineers

Linear Shrinkage
ASTM D4943-02

CLIENT:	Claridge Homes	DEPTH	10' - 12'	FILE NO.:	PG7518
PROJECT:	2380 Tenth Line Road, Ottawa	BH OR TP No:	BH2-25 SS5	DATE SAMPLED	1-May-25
LAB No:	59378	TESTED BY:	CP	DATE RECEIVED	2-May-25
SAMPLED BY:	-	DATE REPORTED:	14-May-25	DATE TESTED	6-May-25

LABORATORY INFORMATION & TEST RESULTS

Moisture	No. of Blows(6)	Calibration (Two Trials)	Tin NO.(P1)	
Tare	4.76	Tin	4.46	4.46
Soil Pat Wet + Tare	64.22	Tin + Grease	4.76	4.76
Soil Pat Wet	59.46	Glass	48.97	48.97
Soil Pat Dry + Tare	40.22	Tin + Glass + Water	85.27	85.27
Soil Pat Dry	35.46	Volume	31.54	31.54
Moisture	**67.68**	**Average Volume**	**31.54**	

Soil Pat + String	35.71
Soil Pat + Wax + String in Air	42.26
Soil Pat + Wax + String in Water	14.77
Volume Of Pat (Vdx)	27.49

RESULTS:

Shrinkage Limit	**35.51**
Shrinkage Ratio	**1.762**
Volumetric Shrinkage	**56.678**
Linear Shrinkage	**13.900**

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

Certificate of Analysis

Report Date: 08-May-2025

Client: Paterson Group Consulting Engineers (Ottawa)

Order Date: 2-May-2025

Client PO: 63009

Project Description: PG7518

	Client ID:	BH2-25 SS3	-	-	-		
	Sample Date:	01-May-25 09:00	-	-	-	-	-
	Sample ID:	2518566-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	70.1	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.66	-	-	-	-	-
Resistivity	0.1 Ohm.m	21.0	-	-	-	-	-
Anions							
Chloride	10 ug/g	19	-	-	-	-	-
Sulphate	10 ug/g	105	-	-	-	-	-

APPENDIX 2

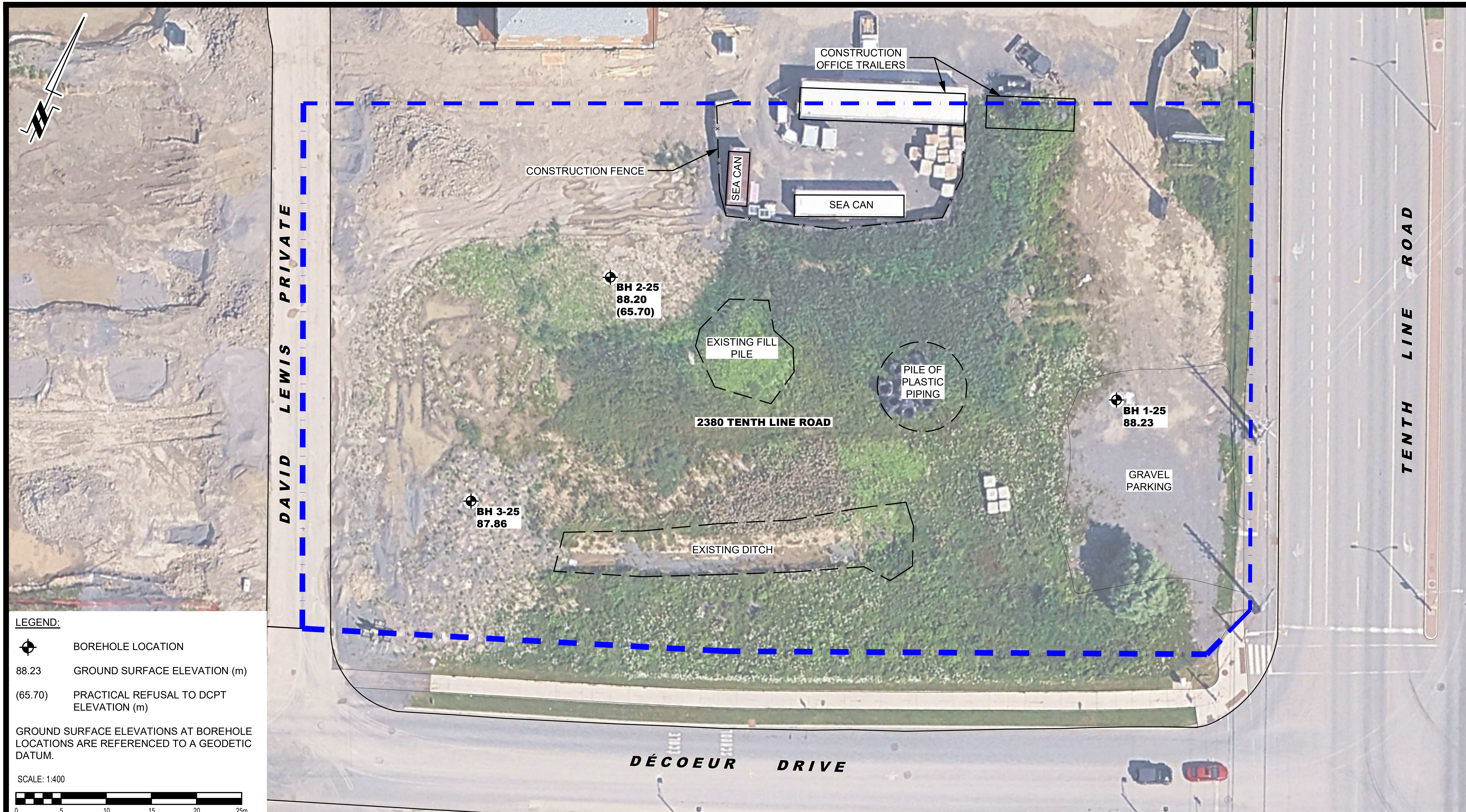
FIGURE 1 – KEY PLAN

DRAWING PG7518-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON GROUP



LEGEND:

⌖ BOREHOLE LOCATION

88.23 GROUND SURFACE ELEVATION (m)

(65.70) PRACTICAL REFUSAL TO DCPT ELEVATION (m)

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

SCALE: 1:400

0 5 10 15 20 25m

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

NO.	REVISIONS	DATE	INITIAL

CLARIDGE HOMES
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
2380 TENTH LINE ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale:	1:400	Date:	05/2025
Drawn by:	ZS	Report No.:	PG7518-1
Checked by:	DR	Dwg. No.:	PG7518-1
Approved by:	SD	Revision No.:	

11x17

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