PATERSON GROUP

Geotechnical Investigation

Proposed Commercial Development

1984 Baseline Road
Ottawa, Ontario

Prepared for Chick-Fil-A Canada ULC

Report PG7643-1 dated August 22, 2025

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Chick-Fil-A Canada ULC to conduct a geotechnical investigation for the proposed commercial development to be located at 1984 Baseline Road in the City of Ottawa (reference should be made to *Figure 1 - Key Plan* included in Appendix 2 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 test holes.

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

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

Investigating the presence or potential presence of contamination on the subject site was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a one-storey commercial building of slab-on-grade construction along the northern portion of the subject site.

It is further understood that associated landscaped areas, asphalt-paved parking areas and drive-thru lanes with landscaped margins are also anticipated surrounding the proposed building. It is understood the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The current geotechnical investigation was conducted on August 1, 2025, and consisted of advancing a total of four (4) boreholes to a maximum depth of 6.7 m below the existing ground surface.

The test hole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground services and available access. The locations of the test holes are shown on Drawing PG7643-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

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

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

Undrained shear strength testing was carried out at regular depth intervals in cohesive soils, using field vanes. Reference should be made to the Soil Profile and Test Data Sheets provided in Appendix 1.

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

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at borehole BH 4-24 for the current field program. 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. Due to the low resistance exerted by the silty clay, the cone was pushed using the hydraulic head of the drill rig until resistance to penetration was encountered. The hammer was then used to further advance the cone to practical refusal.

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 all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the current sampling program. The groundwater observations are discussed in Section 4.3 and presented in the Soil Profile and Test Data Sheets in Appendix 1.

Typical monitoring well construction details are described below:

- Slotted 51 mm diameter PVC screen at the base of each borehole.
- 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- No.3 silica sand backfill within annular space around screen.
- Bentonite hole plug directly above PVC slotted screen.
- Clean backfill from top of bentonite plug to the ground surface.

Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific well construction details.

It should be noted that groundwater observations were also obtained from existing monitoring wells in close proximity to the proposed building. The existing monitoring wells are understood to have been installed by others during previous field investigations. The groundwater observations from the existing monitoring wells are presented in Section 4.3

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations, and the ground surface elevation at each test hole location, were surveyed by Paterson using a GPS unit with respect to a geodetic datum. The locations of the test holes, and ground surface elevation at each test hole location, are presented on Drawing PG7643-1 - Test Hole Location Plan included in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Moisture content testing was completed on all recovered soil samples from the current investigation. Atterberg limits testing was conducted on one (1) selected soil sample and grain-size distribution testing was conducted on three (3) selected soil samples.

The results of the Atterberg limits and grain-size distribution testing are presented in Subsection 4.2 and presented in Appendix 1.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an existing one-storey commercial building immediately surrounded to the north, west, and south with grassed and landscaped areas. An existing concrete loading dock ramp is located along the east side of the existing building. Small to medium trees and bushes were noted on the west and east sides of the existing building. The subject site is bordered to the north by Baseline Road, to the west by Woodroffe Avenue, to the east by an asphalt-paved access lane, and to the south by an asphalt-paved parking lot and an existing one-storey commercial building.

The ground surface across the site is relatively flat, sloping gently from east to west between approximate geodetic elevations 86.3 to 86.8 m. The ground surface at the subject site was generally noted to be approximately 0.6 m above the grade of the surrounding streets.

4.2 Subsurface Profile

Generally, the subsurface profile at the subject site consists of topsoil and/or asphaltic concrete, underlain by a layer of fill, which is underlain by a very stiff brown to firm to stiff grey silty clay deposit. Reference should be made to the Soil Profile and Test Data Sheets in Appendix 1 for details of the soil profile encountered at each borehole location.

DCPT testing was conducted in BH 4-24. Practical refusal to the DCPT was encountered at 9.9 m below ground surface.

Topsoil and Asphalt

The topsoil layer was observed within BH 3-25 and BH 4-25 and was noted to range in thickness between 200 to 240 mm. The asphaltic concrete layer was observed within BH 1-25 and BH 2-25 and was noted to range in thickness between 50 to 75 mm.

Fill

The asphalt layer within BH 1-25 and BH 2-25 was noted to be underlain by a fill layer that was generally observed to consist of a crushed stone asphalt base further underlain by a layer of brown silty sand with variable amounts of crushed stone.

The topsoil layer within BH 3-25 and BH 4-25 was noted to be underlain by a fill layer that was generally observed to consist of brown silty sand with variable amounts of crushed stone. The fill layer throughout the subject site was noted to extend to approximate depths ranging between 0.7 to 3.4 m below the existing ground surface.

Silty Sand

A layer of compact brown silty sand was noted below the fill layer in BH 2-25 and BH 4-25 and was observed to extend to approximate depths ranging between 1.8 and 2.2 m below the existing ground surface.

Silty Clay

The above-noted fill and silty sand layers were observed to be underlain by a native silty clay deposit. The very stiff to stiff, brown silty clay layer was observed to extend to approximate depths ranging between 5.0 to 5.8 m below existing ground surface. The brown silty clay layer was observed to be underlain by a layer of stiff to firm grey silty clay.

Bedrock

Based on available geological mapping, bedrock in the area of the subject site consists of limestone and sandstone of the Rockcliffe Formation with a drift thickness ranging between 10 to 15 m.

Atterberg Limits Testing

Atterberg limits testing was completed on a selected silty clay sample recovered during the current investigation. The results of the Atterberg Limits testing are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1.

Table 1 – Atterberg Limits Results

Borehole	Sample	Depth (m)	LL (%)	PL (%)	PI (%)	Classification
BH 1-25	SS4	2.3 – 2.9	57	25	32	CH

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

Grain Size Distribution and Hydrometer Testing

Grain size distribution analysis was completed on selected recovered samples. The result of the grain size distribution analyses are presented in Table 2 below and on the Grain Size Distribution sheets in Appendix 1.

Table 2 – Grain Size Distribution Analysis Results					
Sample No.	**Depth (m)**	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**
BH 1-25 (SS8)	5.3 – 5.9	0.0	6.8	50.2	43.0
BH 3-25 (SS9)	6.1 – 6.7	0.7	18.5	55.7	25.1
BH 4-25 (SS3)	1.5 – 2.1	0.0	65.2	34.8	

4.3 Groundwater

Groundwater levels were measured in the available monitoring wells throughout the subject site and are presented on the Soil Profile and Test Data sheets in Appendix 1, and in Table 3 below.

Table 3 – Measured Groundwater Levels					
Test Hole Number	**Method**	**Ground Surface Elevation (m)**	**Measured Groundwater Level**		**Date**
			Depth (m)	**Elevation (m)**	
Groundwater Levels Based on Monitoring Wells from Current Investigation					
BH 1-25	Monitoring Well	86.33	3.73	82.60	August 8, 2025
BH 2-25	Monitoring Well	86.61	3.72	82.89	August 8, 2025
BH 3-25	Monitoring Well	86.54	4.18	82.36	August 8, 2025
BH 4-25	Monitoring Well	86.78	4.13	82.65	August 8, 2025
NOTE: The ground surface elevations at the monitoring well locations were surveyed by Paterson using a high precision GPS unit and are referenced to a geodetic datum.					

It should be noted that surface water can become trapped within a backfilled borehole column, which can lead to higher-than-normal groundwater level readings. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, 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. Based on the results of the field investigation, the proposed one-storey commercial building may be founded on conventional spread footings placed on the in-situ, undisturbed, very stiff silty clay bearing surface.

Due to the presence of a silty clay layer, the subject site is subjected to a permissible grade restriction. 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

Subject Site

Topsoil and asphaltic concrete, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. Care should be taken not to disturb subgrade soils during the site preparation activities.

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

Area of Existing Services Below Future Building Footprint

It is anticipated a fill layer will be encountered associated with existing site services will be encountered at the founding depth of the proposed building. Where fill is encountered, provided the in-situ fill is considered to be of relatively workable soils (i.e., compactable using sheepsfoot and/or smooth-drum rollers), consideration could be given to sub-excavating 400 mm below the design founding depth of the proposed structures foundations, proof-rolling (i.e., re-compacting) and reinstating with engineered fill (OPSS Granular A and/or OPSS Granular B Type II crushed stone) as a capping layer for the bearing surface.

Fill Placement

Imported Fill Placement

Fill placed for grading beneath the building footprints should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment as specified in OPSS 501. Fill placed beneath the buildings should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

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

Any soft or poor performing areas should be removed and replaced with engineered fill consisting of OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm. The engineered fill should be placed in maximum 300 mm loose lifts and compacted to 98% SPMDD using suitable vibratory equipment as per OPSS 501.

Site Generated Fill Placement

From a geotechnical perspective, site-generated fill free of organic debris, inorganic material and/or stones/cobbles larger than 200 mm in their longest dimension and sensitive/saturated grey clay soils are generally considered suitable for re-use as pre-grade material throughout the subject site. The site-generated fill may be used for raising the ground surface within the building footprints, above the underside of and around footings, as foundation wall backfill, throughout the proposed paved areas and throughout landscaped areas.

These materials should be spread in a maximum of 300 mm thick loose lifts and at least compacted by the tracks of the spreading equipment to minimize voids below landscaped areas. Clayey workable fill must be compacted using a suitably-sized vibratory sheepsfoot roller.

If this material is to be used to build up the subgrade level for areas to be paved, below the slab-on-grade structure, and below pipe bedding, it should be placed in maximum 300 mm thick loose lifts and compacted using a suitably sized sheepsfoot roller to a minimum of 98% of the material's standard Proctor maximum dry density (SPMDD), in the dry and above-freezing conditions. Each lift of site-generated fill should be reviewed and approved by Paterson field personnel at the time of construction.

Prior to using site-generated soil, topsoil and significant amounts of organics (peat, stumps, logs and/or other organic debris) or deleterious materials should be segregated from the fill prior to re-use. The preparation and segregation of fill material should be reviewed and approved at the time of construction by Paterson personnel. Paterson personnel may advise on the suitability of potential re-use material at that time.

Care will also need to be taken during storage, placement and compaction of the excavated fill and native soils to maintain them in an unfrozen state and at a moisture content which is suitable for compaction. Soils intended for re-use which become frozen and/or which have excessive moisture contents will not be considered suitable for reuse at the subject site. Placement of this material during winter months increases the risk of placing frozen material which may result in future poor performing areas that will require repair.

It should be noted that placement of site-generated fill will be very difficult during the early-spring season, during periods of heavy-rainfall and will not be able to be accomplished during the winter seasons. It is recommended that construction schedules be planned with these constraints considered in the timing and scheduling of these works if this method will be considered.

Any soft or poor performing areas should be removed and replaced with engineered fill consisting of OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm. The engineered fill should be placed in maximum 300 mm loose lifts and compacted to 98% SPMDD using suitable vibratory equipment as per OPSS 501.

5.3 Foundation Design

Bearing Resistance Values – Conventional Spread Footings

Strip footings and pad footings, up to 2 m wide, placed upon an undisturbed, very stiff to stiff 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**.

A geotechnical resistance factor of 0.5 is applied to the above noted bearing resistance values at ULS. The above-noted bearing resistance values at SLS for soil bearing surfaces will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Alternative footing sizes may be able to be accommodated depending on final grading and should be reviewed and advised upon by Paterson during the foundation design and preliminary/detailed design stages. The above-noted footing design bearing resistance values considers the grading restrictions provided in the following section of this report.

Lateral Support

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

Adequate lateral support is provided to a silty clay bearing medium 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 or engineered fill of the same or higher capacity as the bearing soil.

Permissible Grade Raise

Based on the undrained shear strength values of the silty clay deposit encountered throughout the subject site, a permissible grade raise restriction geodetic elevation of **87.20 m** is recommended for grading in close proximity to the proposed building at the subject site. If higher permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures may be advised by Paterson during the preliminary and detailed design stages to mitigate the risks of unacceptable long-term post construction total and differential settlements amongst settlement sensitive structures.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as a **Site Designation X_E** (in accordance with 2024 Ontario Building Code). Soils underlying the subject site are not susceptible to liquefaction or cyclic softening.

Reference should be made to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Slab on Grade Construction

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the native soil subgrade approved by Paterson field personnel at the time of excavation will be considered an acceptable subgrade surface on which to commence backfilling for slab-on-grade construction.

Where the subgrade layer consists of existing crushed stone fill, it is recommended that the existing layer be scarified of the upper 100 mm of existing fill and reinstated with OPSS Granular A crushed stone to a minimum of 98% of the materials SPMDD. The scarified fill surface should be proof-roller using a suitably sized vibratory roller under the supervision of Paterson field personnel.

Any soft areas should be removed and reinstated with an engineered fill, such as OPSS Granular B Type II and compacted to a minimum of 98% of the materials SPMDD. OPSS Granular A, Granular B Type II or Paterson-reviewed and -approved site-generated fill may be used for for backfilling below the floor slab (outside the zone of influence of the footings).

Where existing crushed stone fill is not present or removed due to footing and other construction activities within the building footprint, It is recommended that the upper 200 mm of sub-slab fill consists of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

5.6 Pavement Design

Car only parking areas and access lanes are proposed as part of the development at this site. The proposed pavement structures are shown in Tables 4 and 5 below.

Table 4 – Recommended Pavement Structure – Light Vehicle Parking	
Thickness (mm)	**Material Description**
50	**Wear Course –** Superpave 12.5-FC2 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
300	**SUBBASE –** OPSS Granular B Type II Crushed Stone
SUBGRADE – Either fill, in-situ soil, or sand/crushed stone material placed over in-situ soil	

Table 5 – Recommended Pavement Structure – Local Roadways, Access Lanes and Heavy Vehicle Parking	
Thickness (mm)	**Material Description**
40	**Wear Course** - Superpave 12.5-FC2 Asphaltic Concrete
50	**Upper Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II Crushed Stone
SUBGRADE - Either fill, in situ soil, or sand/crushed stone material placed over the in-situ soil	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

Minimum Performance Graded PG58H-34 asphalt cement should be used for this project. Cement asphalt should be compacted to a minimum average density of 93% and no more than 98%.

Rigid Pavement Structure

It is understood that a rigid pavement structure may be considered for the drive-thru-lane. The rigid pavement structure presented in Table 6 is recommended for the subject drive-through lane for areas where a concrete pad is anticipated.

It should be noted that the reinforced concrete slab will be susceptible to frost heave if frost protection is not provided. Therefore, control and isolation joints are required for the subject concrete slabs.

Table 6 – Rigid Pavement Structure	
Thickness (mm)	**Material Description**
As Specified by Others	**Reinforced Concrete –** Minimum 32 MPa -with 5 to 8% air entrainment
150	**BASE –** OPSS Granular A Crushed Stone
450	**SUBBASE –** OPSS Granular B Type II Crushed Stone
100	**RIGID INSULATION** – High-density extruded polystyrene rigid insulation boards such as DuPont Styrofoam HL-40
SUBGRADE - Either fill, in-situ silty clay or sand/crushed stone material placed over in-situ soil	

To minimize the potential differential frost heave at the interface between the rigid pavement structure and adjacent asphalt pavement structures, a frost taper should be over-excavated below the asphalt pavement structure.

It is recommended that a minimum 600 mm thick frost taper, consisting of a Granular B Type II placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the SPMDD using suitable vibratory equipment, extend horizontally at least 1.5 m beyond the outside edge of the concrete pad.

The frost taper beyond the horizontal section should slope up to match the pavement structure subgrade level at a 3H:1V slope. It is recommended that insulation for frost mitigation is considered below the rigid pavement structure to prevent movement from frost action. For preliminary purposes, it is recommended that 100 mm thick rigid insulation panels extending 1.8 m beyond all faces of the slab are utilized. However, the insulation panel thickness and frost taper extent can be confirmed at the time of grading plan review.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the SPMDD using suitable vibratory equipment.

Full depth isolation joints consisting of approximately 12 mm thick compressible material are recommended adjacent to any existing rigid structure such as curbs, poles, sidewalks, and buildings to allow minor movement to occur independently from each other.

Control joints, also known as contraction joints, provide a location where drying shrinkage cracks or cracking attributed to frost heave can occur without affecting the appearance of the concrete pad. The saw cut control joints should be placed at a minimum 2.4 m grid with a depth of 50 mm and a maximum width of 5 mm.

Re-Use of Existing Crushed Stone Fill

Existing crushed stone fill may be considered for re-use for building up subgrade and as the sub-base layers of paved areas provided the fill does not contain high amounts of fine soil particles (i.e., clays and silts) and generally consists of well-graded crushed stone. The potential re-use material should be reviewed and tested by Paterson field personnel at the time of exposing the material to assess the suitability for re-use and overall quality of the material, since it was not encountered at the time of this investigation.

Pavement Joint Tie-in

Where the proposed pavement structure meets an existing pavement structure, the following recommendations should be followed:

- ❑ A 300 mm wide section of the existing asphalt roadway should be saw cut from the existing pavement edge to provide a sound surface to abut the proposed pavement structure.
- ❑ It is recommended to mill a 300 mm wide and 40 mm deep section of the existing asphalt at the saw cut edge.
- ❑ The proposed pavement structure subbase materials should be tapered no greater than 3H:1V to meet the existing subbase materials.
- ❑ Clean existing granular road subbase materials can be reused upon assessment by Paterson at the time of excavation (construction) as to its suitability.

Light Post Construction

It is expected that light posts will be constructed within the proposed development. Light post pole bases are considered unheated structures and therefore their subgrade would require a minimum of 2.1 m of soil cover or equivalent to be protected from frost action. Generally, it is recommended that the post bases be founded below the depth of frost migration for unheated structures.

Furthermore, it is anticipated that the proposed pole bases are to be installed using open excavation and will be founded upon an undisturbed hard to stiff silty clay. The site excavated material is considered frost susceptible, and not suitable for backfill directly against the bases.

To mitigate potential frost action from the backfill material surrounding the bases, it is recommended that the buried portion of the bases be surrounded by a layer of engineered fill such as OPSS Granular A or Granular B Type II with a minimum thickness of 600 mm surrounding the pole base. This thickness may be reduced to 300 mm if the proposed bases are surrounded by a suitable non-fixed casing material such as a sono-tube shell or other approved product capable of remaining intact below the ground surface.

The pole bases may be founded on a silty clay bearing surface once approved by Paterson at the time of construction. Based on these recommendations, the bearing resistance values noted in Section 5.3 may be applied for end bearing. It should be noted that the pole bases founded on the silty clay deposit will be subject to post-construction settlements of up to 25 mm.

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, 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.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

Given the slab-on-grad nature of the proposed structure, a foundation drainage system is not required for the proposed structure provided foundation backfill recommendations provided below and in Section 5.2 are followed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free draining non-frost susceptible granular materials (such as clean sand or OPSS Granular A or OPSS Granular B Type I or Type II granular material) placed in maximum 300 mm thick loose lifts and compacted using suitably sized compaction equipment. Where backfill supports hardscaping, it is recommended that Paterson review and approve fill placement efforts to ensure adequate benching of lifts upon excavation sidewalls and compaction of each lift.

6.2 Protection Against Frost Action

Foundation Structures

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent thickness of soil cover and foundation insulation, should be provided for adequate frost protection of heated structures.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover, or an equivalent thickness of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of the excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is expected that sufficient room will be available for the greater part of the excavation to be undertake 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 brown and grey clay subsoils at this site are considered to be mainly Type 2 and Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavation side slopes carried out for the building footprint are recommended to be provided surface protection from erosion by rain and surface water runoff if shoring is not anticipated to be implemented. This can be accomplished by covering the entire surface of the excavation side-slopes with tarps secured between the top and bottom of the excavation and approved by Paterson personnel at the time of construction. It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of side-slopes.

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

It is recommended that a trench box be used at all times to protect personnel working in trenches. Based on this, trench boxes should be considered for all sewer pipe installations undertaken throughout the subject site. 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.

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

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.

Reinstatement of the trench located above the pipe cover layer should consist of placing trench-generated workable soil fill (i.e., grey clay is not expected to be workable or re-usable for this purpose) in maximum 300 mm thick loose lifts and compacted using a suitably sized vibratory sheepsfoot roller to a minimum of 95% of the materials SPMDD. Each lift of soil fill placed within the service trenches should be reviewed and approved at the time of construction by Paterson personnel. 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.

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

The clay seals 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. Paterson field personnel should review the placement of all clay seals undertaken at the time of construction.

6.5 Groundwater Control

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

Groundwater Control for Building Construction

An Environmental Activity and Sector Registry (EASR) will be required. A minimum of two to four weeks should be allotted for the completion of the EASR registration and Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg.63/16. Paterson personnel should review the excavation at the time of construction to assess the soil type at the bottom of the excavation and the water infiltration observations, once exposed.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

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

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost into 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.

The trench excavations should be carried out in a manner that will avoid the introduction of frozen materials into the trenches. Also, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. In addition, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure. Additional information could be provided, if required.

Provisions should also be carried out for accommodating spring-thaw conditions when subgrade conditions for pavements and other works are impacted by higher degrees of soil saturation. Additional information should be provided by Paterson for planning winter construction and pavement works.

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 severe, to very aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Considerations

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed in previous investigations by others for the recovered silty clay samples at selected locations throughout the subject site. The soil samples were recovered from elevations below the anticipated design underside of footing elevation. The results of testing by others are presented in Table 1 in Subsection 4.2 and in Appendix 1.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be less than 40% in all the tested clay samples. In addition, based on the moisture level and consistency, the silty clay across the subject site is considered to be a clay of low to medium potential for soil volume change.

The following tree planting setbacks are recommended for the low to medium sensitivity silty clay deposit and where trees are located near buildings founded on cohesive soils. It should be noted that footings supported by a deep foundation consisting of end-bearing piles will not be subject to tree planting setbacks restrictions.

- Large trees (mature height over 14 m) can be planted within these areas provided that a tree to foundation setback equal to the full mature height of the tree can be provided.
- Tree planting setback limits may be reduced to 4.5 m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m), provided that the conditions noted below are met.
- A small tree must be provided with a minimum of 25 m^3 of available soils 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.

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

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 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 the preliminary and detailed grading and servicing plans, from a geotechnical perspective.
- ❑ Review and inspection of the installation of the foundation drainage systems.
- ❑ 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 undertaken by Paterson.

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 Chick-Fil-A Canada ULC, 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.

Nicholas F. R. Versolato, CPI, B.Eng.

Drew Petahtegoose, P.Eng.

LICENSED PROFESSIONAL ENGINEER
August 22, 2025
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Report Distribution:

❑ Chick-Fil-A Canada ULC
❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERGS TESTING RESULTS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1984 Baseline Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 362640.32 **NORTHING:** 5023747.20 **ELEVATION:** 86.33

PROJECT: Proposed Commercial Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: August 1, 2025

FILE NO. : PG7643

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION (GROUND SURFACE)	Depth / Elevation
ASPHALT	0.05m [86.28m]
FILL: Crushed stone	0.91m [85.42m]
FILL: Brown silty sand	1.22m [85.11m]
Very stiff, brown **SILTY CLAY**	5.03m [81.30m]
Firm, grey **SILTY CLAY**	6.71m [79.62m]
End of Borehole	

(GWL at 3.73 m depth - August 8, 2025)

SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)	REMOULDED SHEAR STRENGTH (kPa)	UNDRAINED SHEAR STRENGTH (kPa)
AU 1			4		
SS 2	87	4-4-3-3 7	17 34		
SS 3	96	4-4-4-6 8	25		
SS 4	100	P	37		>121
SS 5	92	P	41		>121
SS 6	100	P	39	17	116
SS 7	100	P	40 39	14	101
SS 8	100	P	49		34
SS 9	87	P	50		39

■ 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

MONITORING WELL CONSTRUCTION: 3.73 m 2025-08-08; 6.71m

DEPTH (m): 1 2 3 4 5 6 7 8 9 10 11 12

ELEVATION (m): 86 85 84 83 82 81 80 79 78 77 76 75

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1984 Baseline Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 362608.38 **NORTHING:** 5023732.15 **ELEVATION:** 86.61

PROJECT: Proposed Commercial Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: August 1, 2025

FILE NO. : PG7643

HOLE NO. : BH 2-25

SAMPLE DESCRIPTION — GROUND SURFACE

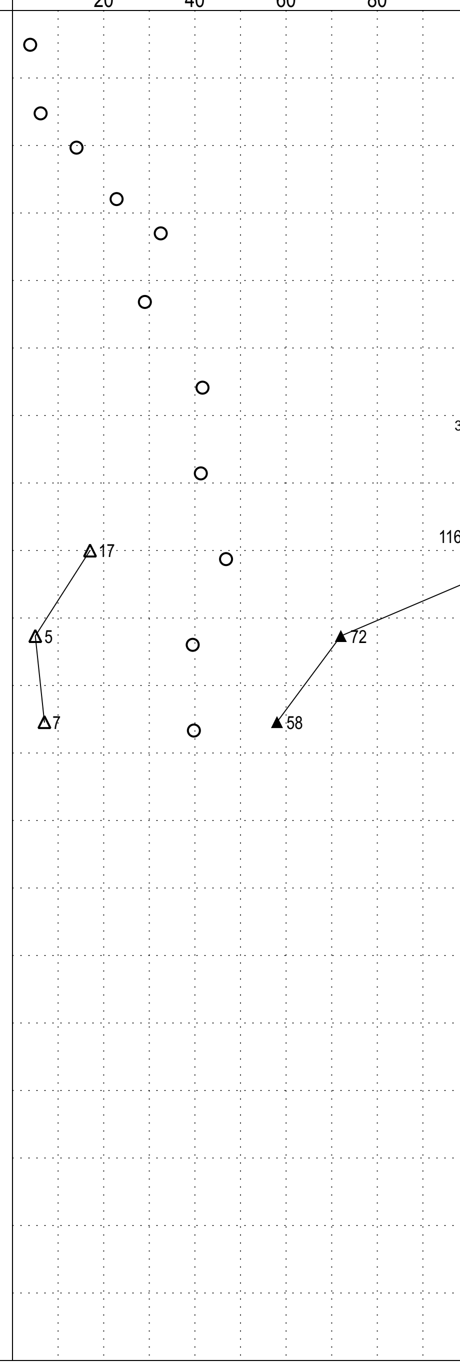
Sample Description	Depth / Elevation
ASPHALT	0.08m [86.53m]
FILL: Crushed stone, with silty sand	0.69m [85.92m]
Compact, brown **SILTY SAND**	1.83m [84.78m]
Very stiff, brown **SILTY CLAY**	5.26m [81.35m]
Stiff, grey **SILTY CLAY**	6.71m [79.90m]
End of Borehole (GWL at 3.72 m depth - August 8, 2025)	

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			4
SS 2	42	6-12-11-11 23	6 14
SS 3	67	3-3-4-6 7	23 33
SS 4	100	2-2-3-3 5	29
SS 5	100	2-2-3-2 5	42
SS 6	100	2-2-2-2 4	41
SS 7	100	P	47
SS 8	92	P	40
SS 9	71	P	40

■ 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



MONITORING WELL CONSTRUCTION

3.72 m 2025-08-08

6.71m

DEPTH (m): 1 2 3 4 5 6 7 8 9 10 11 12

ELEVATION (m): 86 85 84 83 82 81 80 79 78 77 76 75

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

1984 Baseline Road, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 362629.46 **NORTHING:** 5023781.93 **ELEVATION:** 86.54

PROJECT: Proposed Commercial Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: August 1, 2025

FILE NO. : PG7643

HOLE NO. : BH 3-25

SAMPLE DESCRIPTION	Depth / Elevation
GROUND SURFACE	
TOPSOIL	0.20m [86.34m]
FILL: Brown silty sand, with gravel and crushed stone	1.07m [85.47m]
FILL: Brown silty clay, trace crushed stone and topsoil	3.35m [83.19m]
Very stiff, brown **SILTY CLAY**	5.26m [81.28m]
Stiff, grey **SILTY CLAY**	6.71m [79.83m]
End of Borehole	
(GWL at 4.18 m depth - August 8, 2025)	

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			10
SS 2	8	6-8-5-5 13	14
SS 3	42	2-2-3-3 5	14
SS 4	83	1-2-3-2 5	32
SS 5	67	1-1-3-3 4	35 34
SS 6	92	2-2-2-2 4	45
SS 7	100	P	37
SS 8	71	P	55
SS 9	79	P	32

■ PEN. RESIST. (BLOWS/0.3m)
DCPT (50mm DIA. CONE)
20 40 60 80

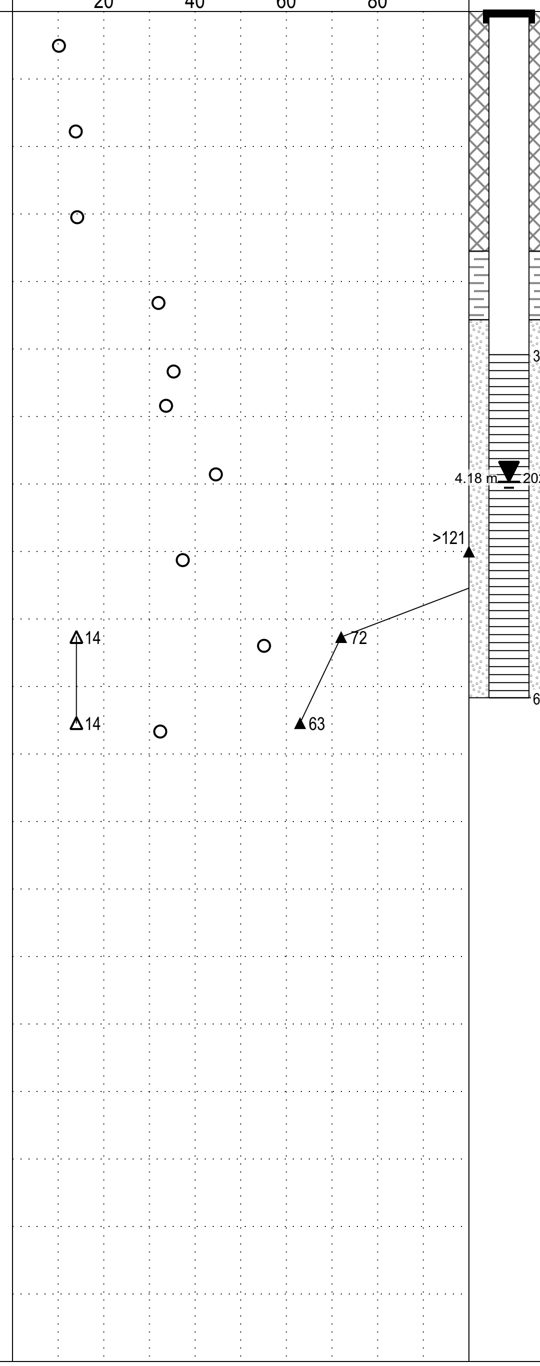
△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa)
20 40 60 80

PL (%) WATER CONTENT (%) LL (%)
20 40 60 80

MONITORING WELL CONSTRUCTION

DEPTH (m): 1 2 3 4 5 6 7 8 9 10 11 12

ELEVATION (m): 86 85 84 83 82 81 80 79 78 77 76 75



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SOIL PROFILE AND TEST DATA

PATERSON GROUP

Geotechnical Investigation
1984 Baseline Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 362597.92 **NORTHING:** 5023764.34 **ELEVATION:** 86.78

PROJECT: Proposed Commercial Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: August 1, 2025

FILE NO. : PG7643

HOLE NO. : BH 4-25

SAMPLE DESCRIPTION	DEPTH / ELEVATION
GROUND SURFACE	
TOPSOIL	0.25m [86.53m]
FILL: Brown silty sand, with crushed stone and clay	1.45m [85.33m]
Compact, brown **SILTY SAND**, trace clay	2.21m [84.57m]
Very stiff, brown **SILTY CLAY**	5.79m [80.99m]
Stiff, grey **SILTY CLAY**	6.71m [80.07m]
Dynamic Cone Penetration Test commenced at 6.71 m depth	9.88m [76.90m]
End of Borehole	

DCPT pushed from 6.71 m to 9.45 m depth

Practical refusal to DCPT at 9.88 m depth

(GWL at 4.13 m depth - August 8, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			8
SS 2	58	4-4-3-6 7	12
SS 3	46	3-4-5-5 9	19
SS 4	100	2-5-5-6 10	34
SS 5	67	3-3-3-4 6	41
SS 6	100	2-1-2-2 3	47
SS 7	100	P	33
SS 8	100	P	41
SS 9	71	P	41

■ 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

>121

>121

△10 ▲58

■67

100

MONITORING WELL CONSTRUCTION: 3.66m; 4.13 m 2025-08-08; 6.71m

ELEVATION (m): 86, 85, 84, 83, 82, 81, 80, 79, 78, 77, 76, 75

DEPTH (m): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12

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

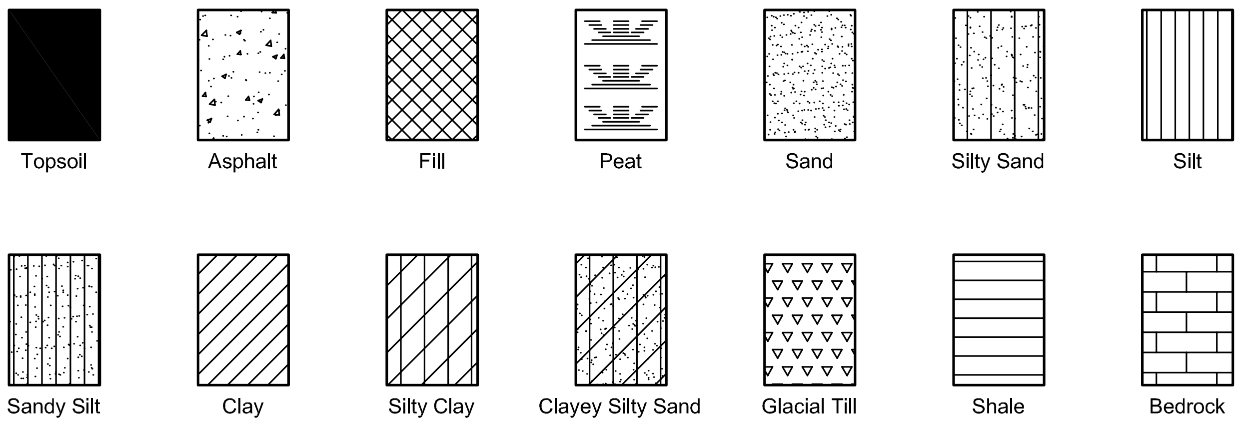
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

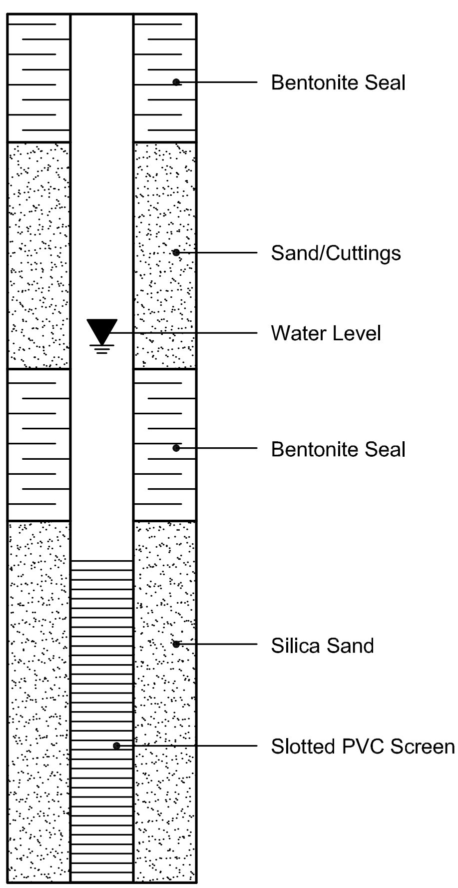
SYMBOLS AND TERMS (continued)

STRATA PLOT

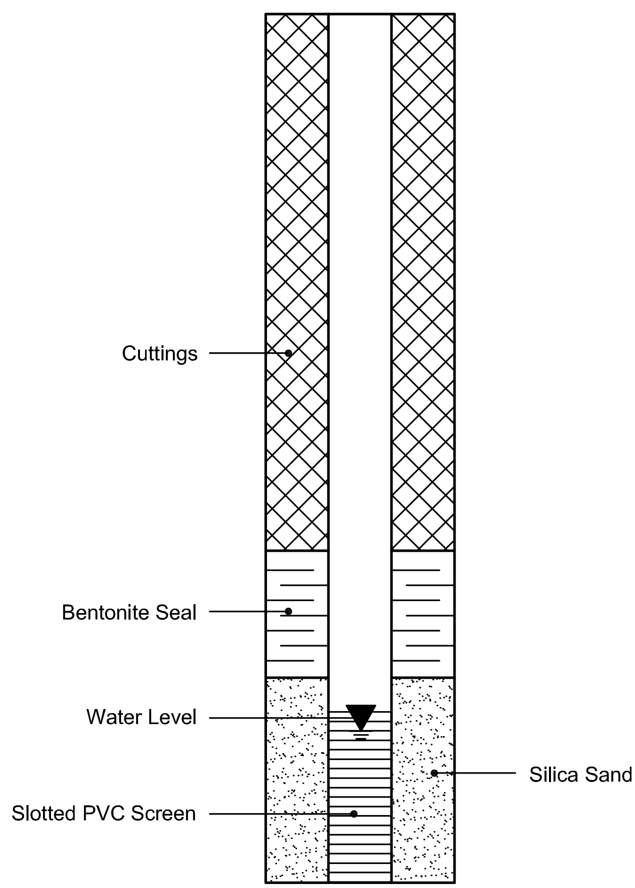


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG7643 - 1 - TEST HOLE LOCATION PLAN

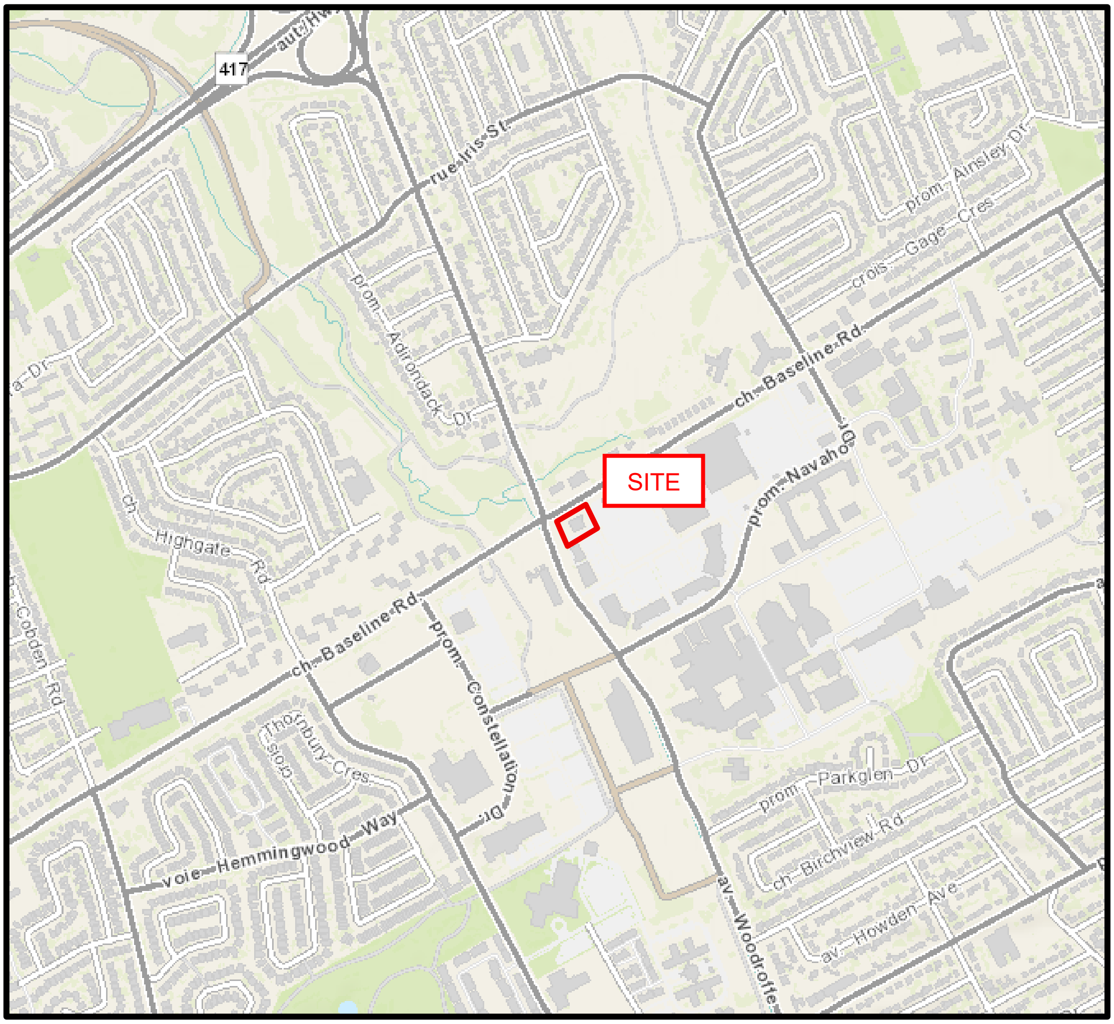
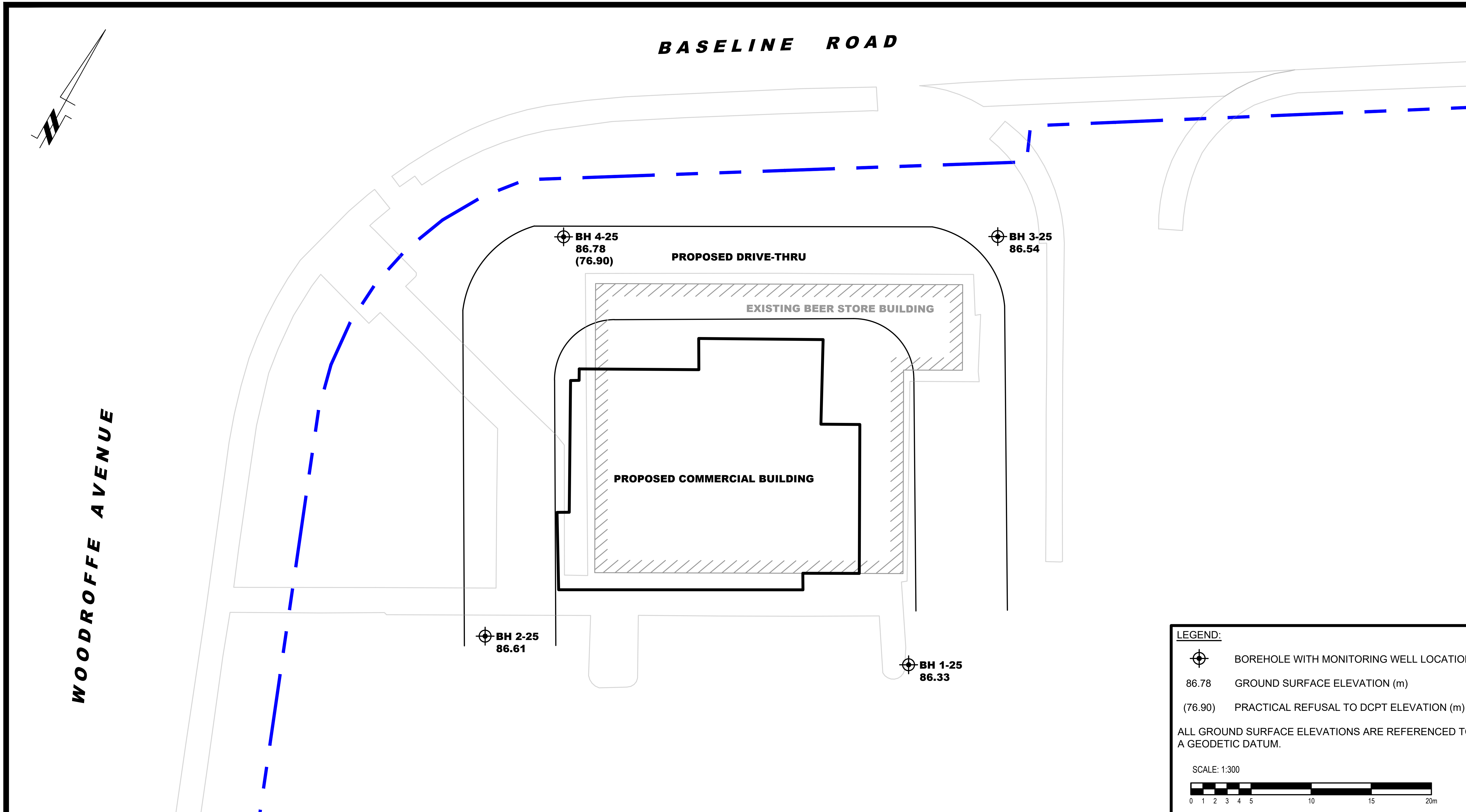


FIGURE 1

KEY PLAN

PATERSON GROUP



LEGEND:

⊕	BOREHOLE WITH MONITORING WELL LOCATION
86.78	GROUND SURFACE ELEVATION (m)
(76.90)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)

ALL GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:300

0 1 2 3 4 5 10 15 20m

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

NO.	REVISIONS	DATE	INITIAL

CHICK-FIL-A CANADA ULC

GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
1984 BASELINE ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:300	Date: 07/2025
Drawn by: NFRV	Report No.: PG7643-1
Checked by: NFRV	Dwg. No.: PG7643-1
Approved by: DP	Revision No.:

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