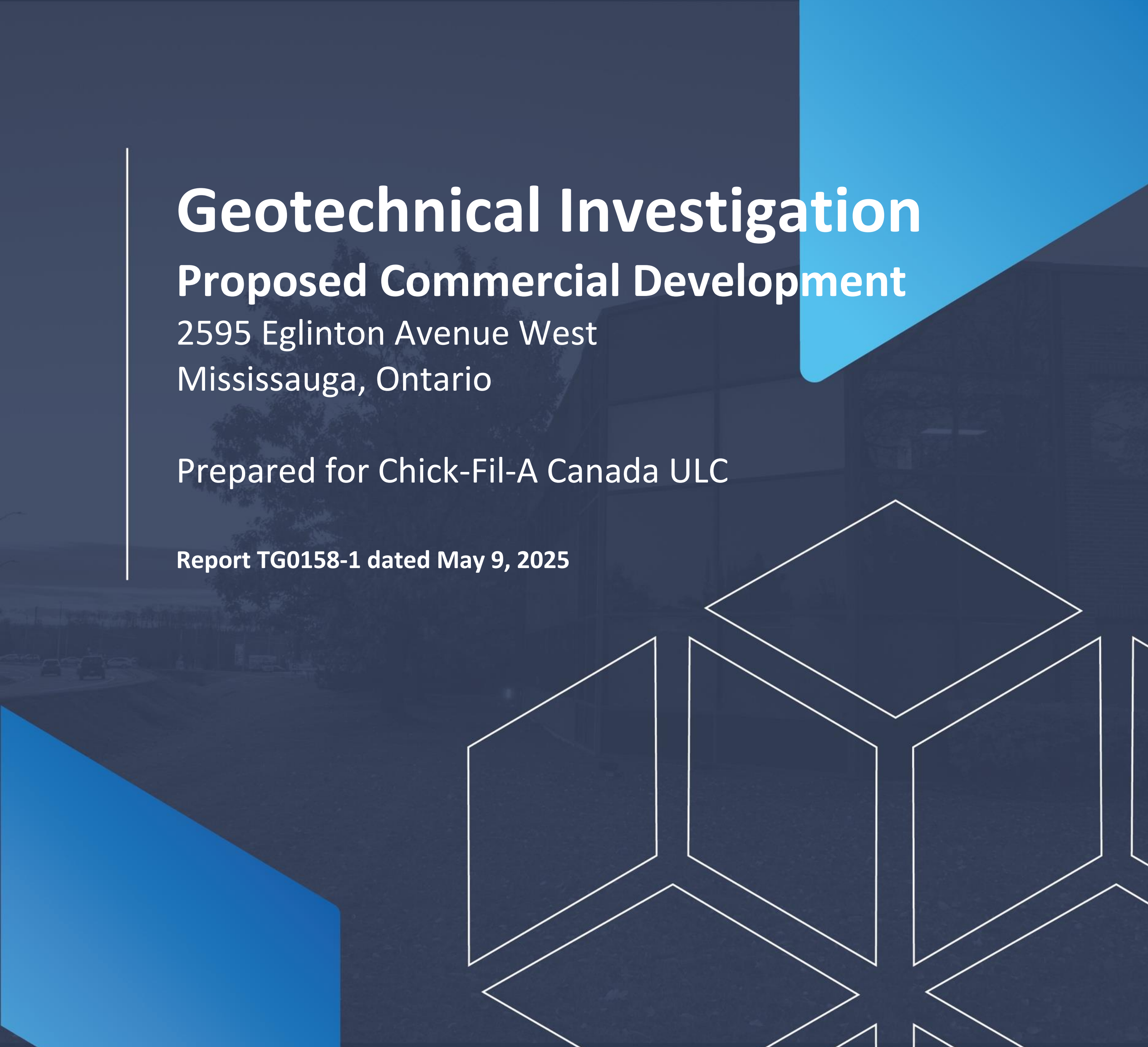
PATERSON GROUP



Geotechnical Investigation

Proposed Commercial Development

2595 Eglinton Avenue West
Mississauga, Ontario

Prepared for Chick-Fil-A Canada ULC

Report TG0158-1 dated May 9, 2025

Table of Contents

Appendices

Appendix 1	Soil Profile and Test Data Sheets Symbols and Terms Analytical Testing Results
Appendix 2	Figure 1 – Key Plan Drawing TG0158-1 - Test Hole Location Plan

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 2595 Eglinton Avenue West in the City of Mississauga (reference should be made to *Figure 1 - Key Plan* 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 western portion of the subject site.

It is further understood that associated landscaped areas, asphalt-paved parking areas and access lanes with landscaped margins are also anticipated surrounding the proposed building, including a portion of the drive-thru lane being provided with a roof structure. 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 April 30, 2025, and consisted of a total of five (5) boreholes advanced to a maximum depth of 6.2 m below the existing grade. 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 TG0158-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck-mounted 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.

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. 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 5-25 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.

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 BH 5-25, and plastic pipe piezometers were installed in all other boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

The groundwater observations are further discussed in Section 4.3 and are presented in the Soil Profile and Test Data Sheets in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities.

The test hole locations, and 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 TG0158-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

Moisture content testing was completed on all recovered soil samples from the current investigation. A total of two (2) grain-size distribution testing was completed on selected soil samples. The results 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

Overall Parcel

The subject site is currently a combination of paved parking and vacant grassed areas. The northern and eastern portions of the site is occupied by a paved parking area exhibiting surface cracking and patched areas. The southern half consists of an unpaved, grass-covered area with a mature tree. The ground surface is relatively flat, sloping gently toward the southwest, and is at-grade with the surrounding area. The site is bounded to the north and west by landscaping and access roads for the Erin Mills Town Center, to the east by an asphalt-paved parking lot serving an adjacent commercial building and to the south by Eglinton Avenue West.

Existing Pavement

The existing asphalt pavement was generally observed to be in a moderate condition. The pavement exhibits some concentrated areas of alligator cracking and relatively long longitudinal cracking throughout the entire parking area. Crack sealing was noted at the majority of cracks which indicates active maintenance of the pavement. The asphalt surface surrounding some catch-basins appeared to be highly fractured and slightly sunken below the surrounding ground surface and lid. This indicates potentially poor compaction of catch-basins backfill while the subgrade for the structure is suitable. Overall, the pavement is in moderate condition and there are no major deficiencies observed during our cursory review.

4.2 Subsurface Profile

Generally, the subsurface profile at the subject site consists of topsoil and/or asphaltic concrete, underlain by a reddish brown compact to very dense glacial till deposit. It is expected that a granular fill layer ranging from 0.2 to 0.5 m is underlying the asphaltic concrete.

The topsoil layer was observed to be approximately 75 mm thick followed by a glacial till deposit extending a minimum of 4.6 to 6.2 m from the ground surface. The glacial till matrix was observed to consist of compact to very dense reddish brown clayey silt mixed with shale fragments, cobbles and gravel. DCPT testing was conducted in BH 5-24. Practical refusal to the DCPT was encountered at 6.3 m below ground surface.

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.

Bedrock

Based on available geological mapping and Ontario well records, bedrock in the area of the subject site consists of interbedded shale, limestone, dolostone and siltstone of the Queenston Formation with a drift thickness ranging between 1 to 6 m.

Grain Size Distribution and Hydrometer Testing

Grain size distribution analysis was completed on a selected recovered sample and is presented in Table 1 below.

Table 1 – Grain Size Distribution Analysis Results

Sample No.	Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 1-25 SS1	0.8 - 1.4	0.0	5.5	84.5	10.0
BH 5-25 SS2	1.5 -2.1	1.7	8.1	80.3	10.0

4.3 Groundwater

A monitoring well was installed in BH 5-25, and plastic pipe piezometers were installed in the remaining boreholes. Water level measurements were recorded at all test hole locations on May 8, 2025. The observed water levels are shown in Table 2 below and are presented in the Soil Profile and Test Data Sheets in Appendix 1.

Table 2 – Summary of Water Level Readings

Test Hole ID	Method	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-25	Piezometer	176.92	Dry	N/A	May 8, 2025
BH 2-25	Piezometer	177.00	Dry	N/A	
BH 3-25	Piezometer	177.13	Dry	N/A	
BH 4-25	Piezometer	176.95	Dry	N/A	
BH 5-25	Monitoring Well	176.98	5.45	171.53	

Note: The ground surface elevation at each test hole location was surveyed using a handheld GPS unit and references to a geodetic datum.

It should be noted that the groundwater level is subject to seasonal fluctuations. Therefore, groundwater 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, compact to very dense glacial till bearing surface.

Due to the presence of the glacial till layer, all contractors should be prepared for the removal of boulders during building construction and servicing installation.

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 throughout portions 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 300 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

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).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids.

If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in maximum 225 mm thin lifts to a minimum density of 95% SPMDD using suitable compaction equipment as per OPSS 501.

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

Footings placed on an undisturbed, compact to very dense glacial till or on engineered fill pads placed directly over the approved native soil can be designed using a bearing resistance value at serviceability limit states (SLS) of **300 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **450 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.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Above the groundwater level, adequate lateral support is provided to the in-situ bearing medium soils when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in-situ soil or engineered fill of the same or higher capacity as that of the bearing medium.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Site Class C** (in accordance with 2012 Ontario Building Code) or a **Site Designation X_C** (in accordance with 2024 Ontario Building Code). Soils underlying the subject site are not susceptible to liquefaction. 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.

For structures with slab-on-grade construction, 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 structures should be placed in a maximum of 300 mm thick loose layers and compacted to a minimum of 98% of the material’s SPMDD.

Where the subgrade consists of loose glacial till, a vibratory drum roller should complete several passes over the subgrade surface in an effort to re-compact the in-situ soils, and under the supervision and approval of Paterson personnel. Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill and as advised and approved by Paterson personnel. OPSS Granular A or Granular B Type II crushed stone are recommended for backfilling below the floor slab.

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 3 and 4 below.

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 crushed stone. 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 PG58-34 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%.

Table 3 – Recommended Pavement Structure – Light Vehicle Parking

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 Crushed Stone
SUBGRADE – Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill.	

Table 4 – Recommended Pavement Structure – Access Lanes and Heavy Truck Parking/Loading Areas

Thickness (mm)	Material Description
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Upper Binder Course** – HL-8 or 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 OPSS Granular B Type I or II material placed over in situ soil or fill.	

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 5 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 5 – Rigid Pavement Structure for Drive-Thru Concrete Apron

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
300	**SUBBASE** – OPSS Granular B Type II Crushed Stone
75	**RIGID INSULATION** – High-density extruded polystyrene rigid insulation boards such as DuPont Styrofoam HL-40
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill.	

The rigid insulation layer is recommended to extend a minimum of 1.8 m horizontally in all directions beyond the footprint of the concrete apron. 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 and long frost taper, consisting of a OPSS Granular B Type II placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the materials SPMDD be located at the edge of the insulation layer extending within below paved areas. Beyond the 600 mm long frost-taper sub-excavation, the sub-excavation should slope up to match the pavement structure subgrade level at a 3H:1V slope.

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 base and subbase stone 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 1.5 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 compact to very dense glacial till. 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 500 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 glacial till 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 glacial till deposit will be subject to post-construction settlements of up to 25 mm.

Concrete and Brick Sidewalks Adjacent to Buildings

It is recommended all hardscaping (i.e., sidewalks and concrete pathways) surrounding the buildings be underlain by a minimum of 150 mm of OPSS Granular A and further by 450 mm of OPSS Granular B Type II crushed stone placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of the materials SPMDD.

The subgrade should consist of either in-situ, undisturbed soils, or re-compacted site-generated fill that had been placed in 300 mm thick loose lifts and compacted to a minimum of 95% of the materials SPMDD during the foundation wall backfilling efforts. The engineered fill layers should be placed on appropriately prepared subgrade and foundation wall backfill as identified herein and in Section 5.2 of this report.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

Given the nature of the in-situ subsoils and proposed slab-on-grade structure, a perimeter foundation drainage system is not considered required for the proposed structure.

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 B Type I granular material) or site-generated workable soils placed in maximum 300 mm thick loose lifts and compacted using suitably sized compaction equipment. Where backfill support 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 1.2 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 1.5 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 glacial till 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 Construction Specification for Trenching, Backfilling, and Compacting (OPSS.PROV 401, November 2015) from the Ontario Provincial Standards for Roads and Public Works (OPS).

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The bedding should extend to the spring line of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material’s standard Proctor maximum dry density.

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 a maximum of 225 mm thick lifts and compacted to 99% of the SPMDD.

It should generally be possible to re-use the site generated fill materials (moist, not wet) above the cover material if excavation and filling operations are carried out in dry and non-freezing weather conditions. Wet/saturated glacial till subsoils, if encountered, should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used. Cobbles and boulders greater than 150 mm in diameter should be segregated from the fill prior to re-use to ensure adequate compaction levels can be achieved.

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 300 mm thick loose lifts and compacted to a minimum of 95% of the material’s SPMDD.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavation should be low to moderate and 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.

Permit to Take Water

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) is not anticipated but 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 Persons as stipulated under O.Reg. 63/16.

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

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The 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 moderate, to aggressive corrosive environment.

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.

LICENSED PROFESSIONAL ENGINEER
May 9, 2025
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Balaji Nirmala, M.Eng.

Drew Petahtegoose, P.Eng.

Report Distribution:

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

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2595 Eglinton Avenue West, Mississauga, Ontario

COORD. SYS.: UTM ZONE 17 **EASTING:** 604244.89 **NORTHING:** 4823472.18 **ELEVATION:** 176.92

PROJECT: Proposed Commercial Building

ADVANCED BY: Truck Mounted Drill Rig & Hydrovac truck

REMARKS: Hydro excavation to 1.88 m

DATE: April 30, 2025

FILE NO. : TG0158

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL 0.08m [176.84m]

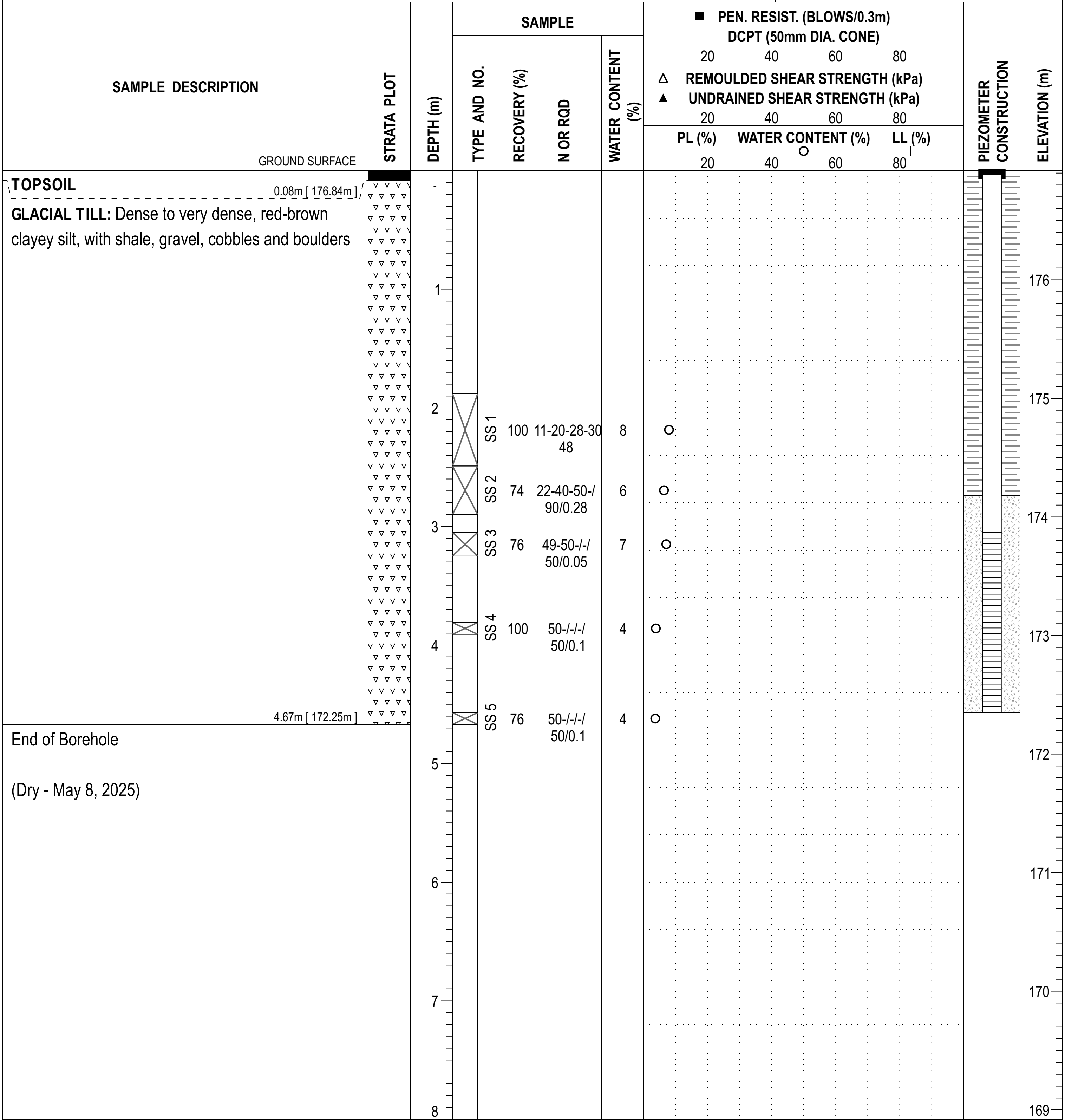
GLACIAL TILL: Dense to very dense, red-brown clayey silt, with shale, gravel, cobbles and boulders

4.67m [172.25m]

End of Borehole

(Dry - May 8, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
SS 1	100	11-20-28-30 48	8
SS 2	74	22-40-50-/ 90/0.28	6
SS 3	76	49-50-/-/ 50/0.05	7
SS 4	100	50-/-/-/ 50/0.1	4
SS 5	76	50-/-/-/ 50/0.1	4



P:/Autocad Drawings/Test Hole Data Files/TG01xx/TG0158/data.sqlite 2025-05-09, 10:41 Paterson_Template JM

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

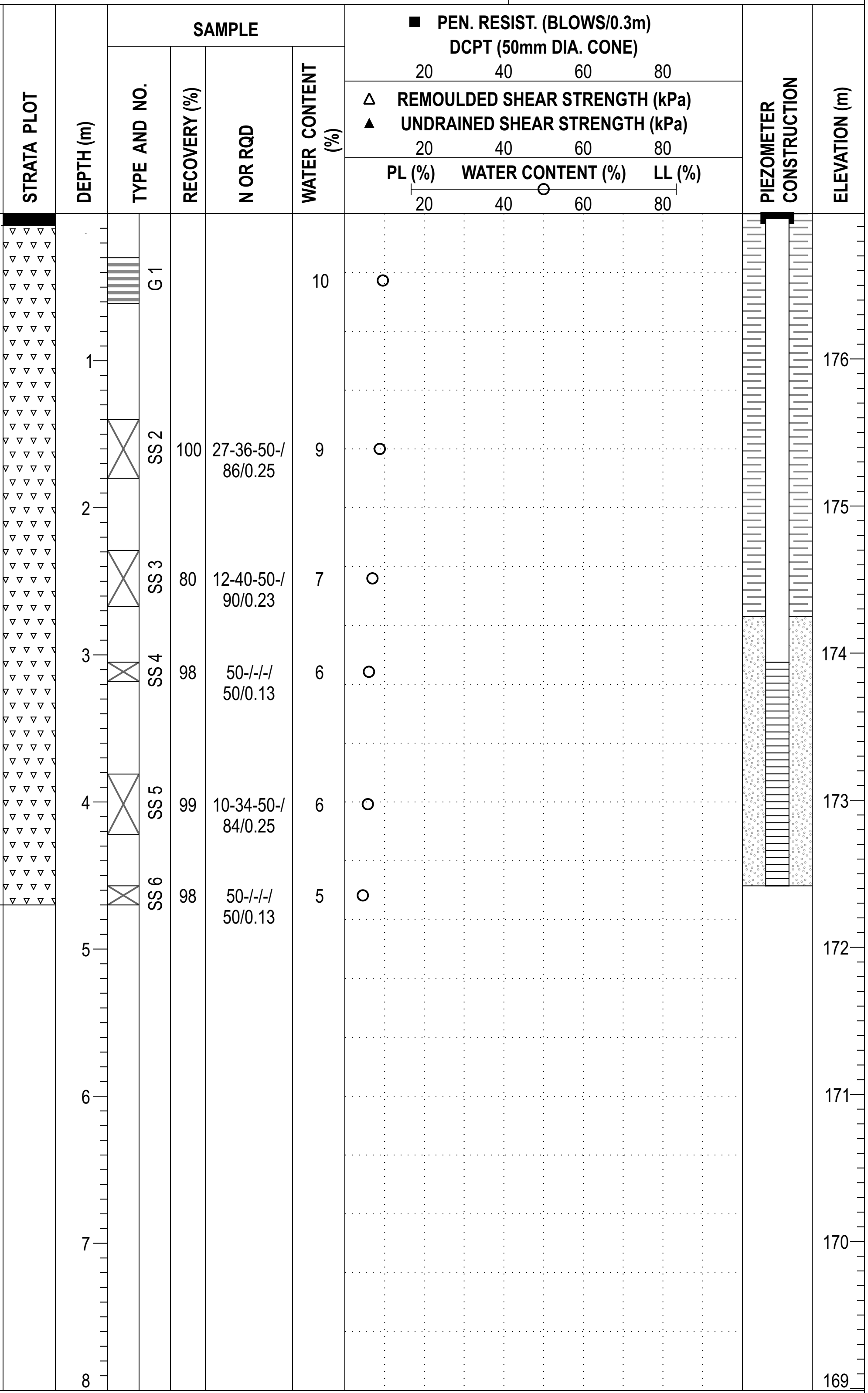
2595 Eglinton Avenue West, Mississauga, Ontario

COORD. SYS.: UTM ZONE 17 **EASTING:** 604237.56 **NORTHING:** 4823464.12 **ELEVATION:** 176.99

PROJECT: Proposed Commercial Building
ADVANCED BY: Truck Mounted Drill Rig & Hydrovac truck
REMARKS:
DATE: April 30, 2025

FILE NO. : TG0158
HOLE NO. : BH 2-25

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
GROUND SURFACE					
TOPSOIL 0.08m [176.91m]					
GLACIAL TILL: Dense to very dense, red-brown clayey silt, with shale, gravel, cobbles and boulders		G 1			10
		SS 2	100	27-36-50-/ 86/0.25	9
		SS 3	80	12-40-50-/ 90/0.23	7
		SS 4	98	50-/-/-/ 50/0.13	6
		SS 5	99	10-34-50-/ 84/0.25	6
4.70m [172.29m]		SS 6	98	50-/-/-/ 50/0.13	5
End of Borehole (Dry - May 8, 2025)					



P:/Autocad Drawings/Test Hole Data Files/TG01xx/TG0158/data.sqlite 2025-05-09, 10:41 Paterson_Template JM

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2595 Eglinton Avenue West, Mississauga, Ontario

COORD. SYS.: UTM ZONE 17 **EASTING:** 604229.54 **NORTHING:** 4823476.59 **ELEVATION:** 177.13

PROJECT: Proposed Commercial Building

ADVANCED BY: Truck Mounted Drill Rig & Hydrovac truck

REMARKS:

DATE: April 30, 2025

FILE NO. : TG0158

HOLE NO. : BH 3-25

SAMPLE DESCRIPTION

GROUND SURFACE

TOPSOIL 0.08m [177.05m]

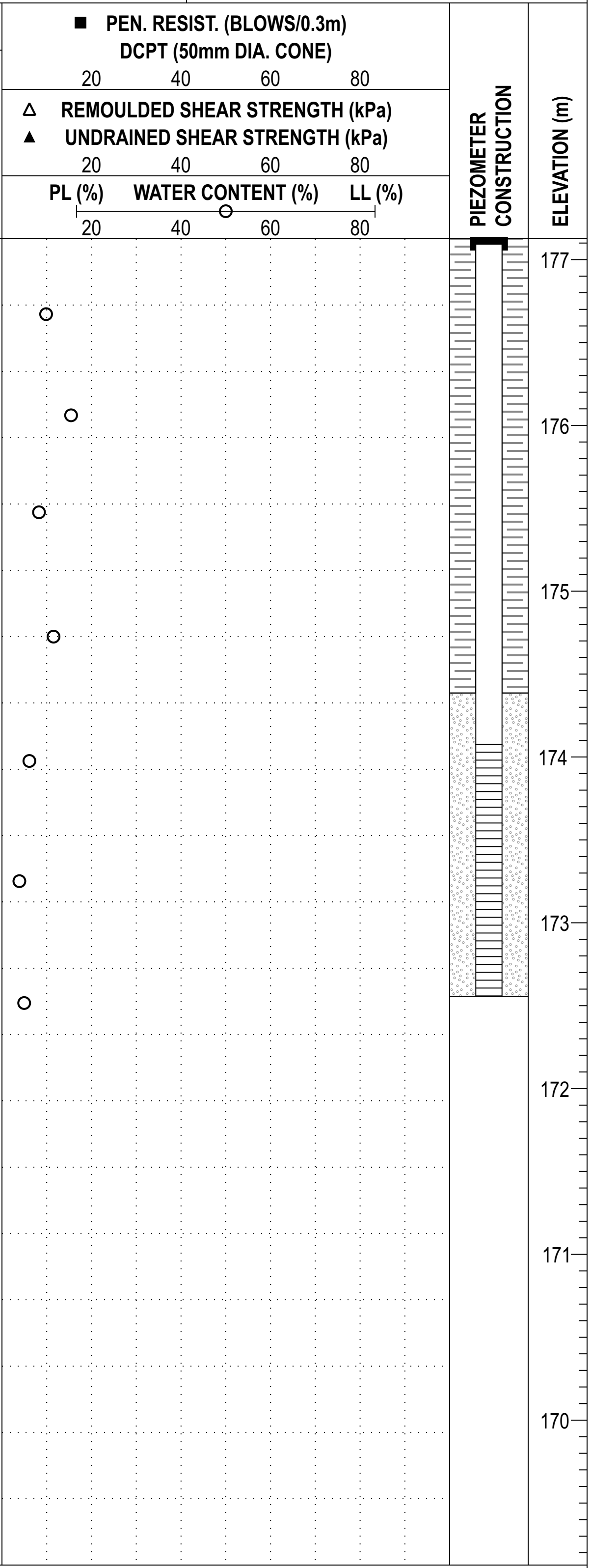
GLACIAL TILL: Compact to very dense, red-brown clayey silt, with shale, gravel, cobbles and boulders

4.65m [172.48m]

End of Borehole

(Dry - May 8, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
G 1			10
SS 2	100	9-8-9-13 17	15
SS 3	98	17-50-/-/ 50/0.1	8
SS 4	100	11-50-/-/ 50/0.08	12
SS 5	100	20-50-/-/ 50/0.05	6
SS 6	98	50-/-/-/ 50/0.13	4
SS 7	95	50-/-/-/ 50/0.08	5



P:/Autocad Drawings/Test Hole Data Files/TG01xx/TG0158/data.sqlite 2025-05-09, 10:41 Paterson_Template JM

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2595 Eglinton Avenue West, Mississauga, Ontario

COORD. SYS.: UTM ZONE 17 **EASTING:** 604225.57 **NORTHING:** 4823488.81 **ELEVATION:** 176.95

PROJECT: Proposed Commercial Building

ADVANCED BY: Truck Mounted Drill Rig & Hydrovac truck

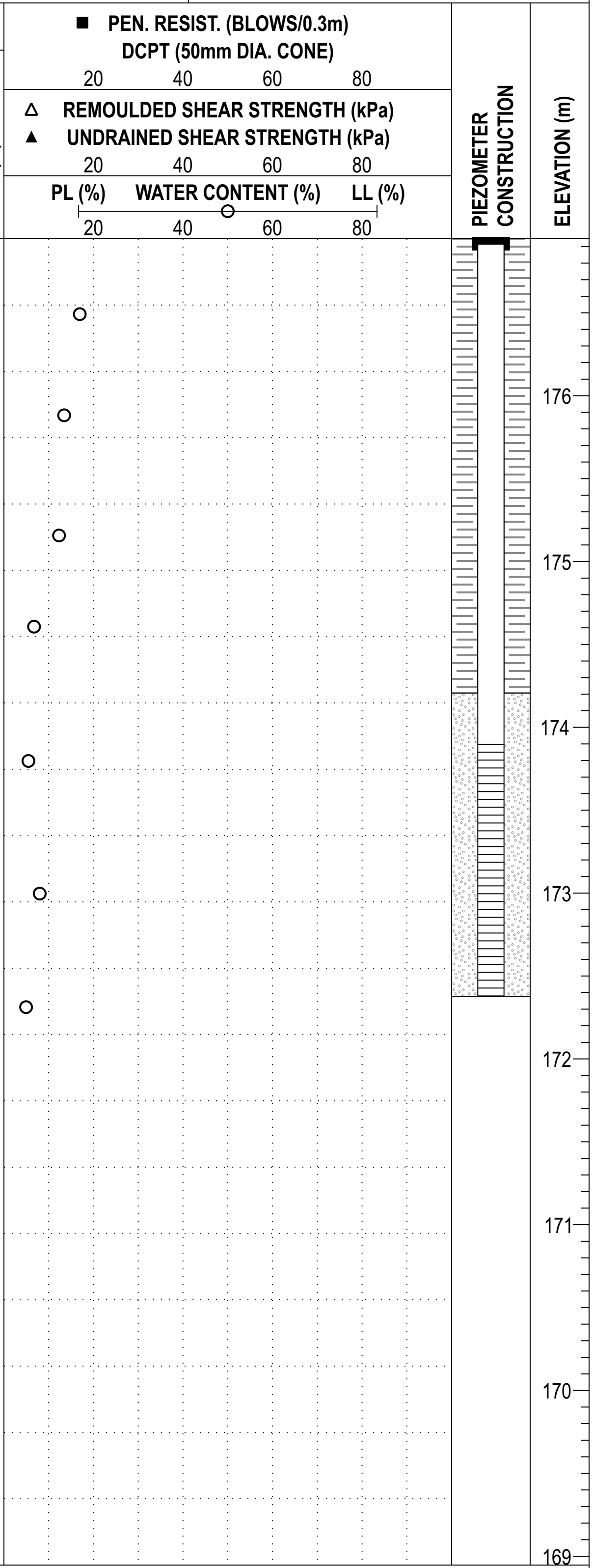
REMARKS:

DATE: April 30, 2025

FILE NO. : TG0158

HOLE NO. : BH 4-25

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
GROUND SURFACE					
TOPSOIL 0.08m [176.87m]					
GLACIAL TILL: Compact to very dense, red-brown clayey silt, with shale, gravel, cobbles and boulders		G 1			17
	1	SS 2	100	7-9-9-28 18	13
	2	SS 3	94	10-21-41-50 62	12
		SS 4	100	50-/-/-/ 50/0.1	7
	3	SS 5	88	48-50-/-/ 50/0.05	5
	4	SS 6	100	17-50-/-/ 50/0.13	8
4.70m [172.25m]		SS 7	98	50-/-/-/ 50/0.13	5
End of Borehole (Dry - May 8, 2025)	5				
	6				
	7				
	8				



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

2595 Eglinton Avenue West, Mississauga, Ontario

PATERSON GROUP

COORD. SYS.: UTM ZONE 17 **EASTING:** 604217.30 **NORTHING:** 4823476.87 **ELEVATION:** 176.98

PROJECT: Proposed Commercial Building

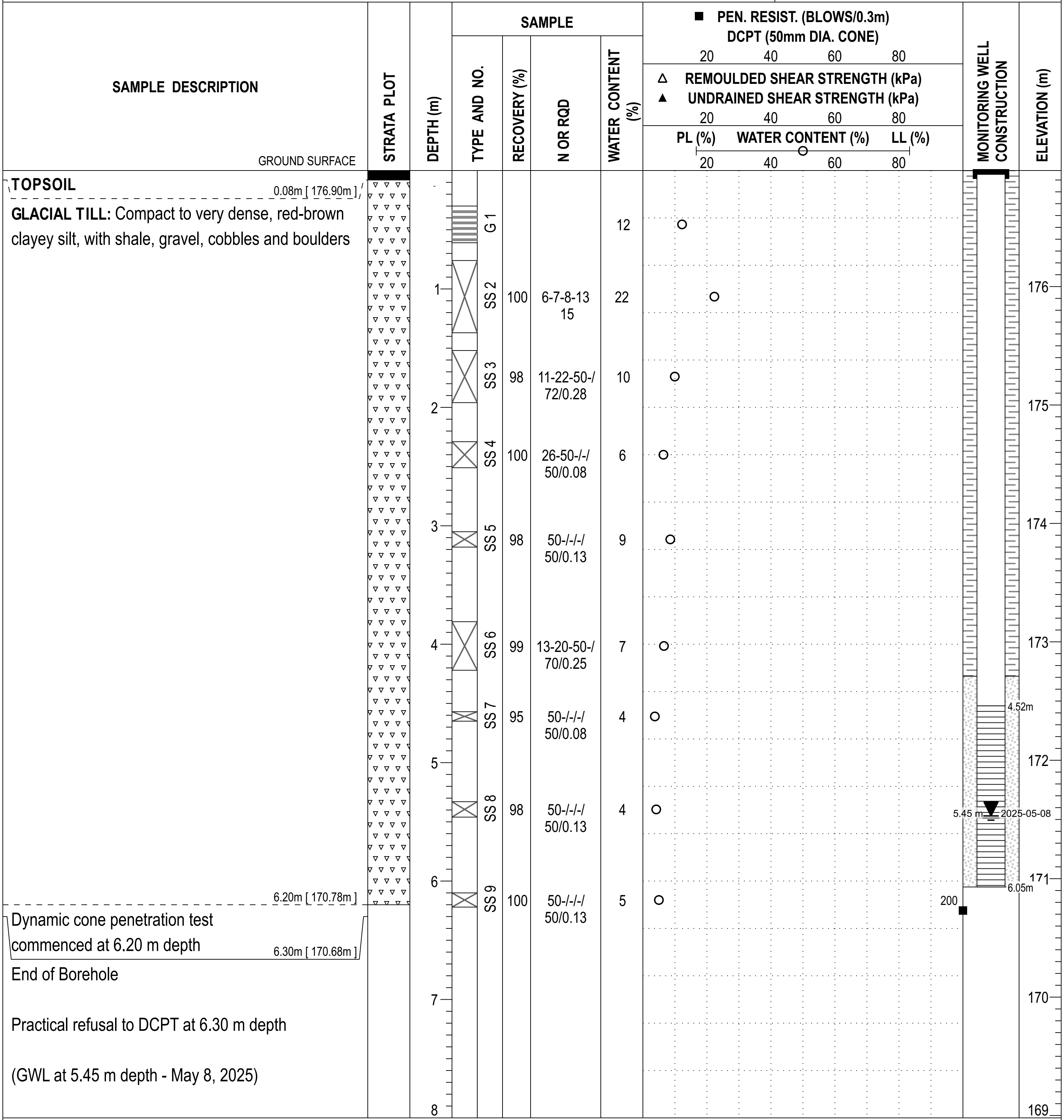
ADVANCED BY: Truck Mounted Drill Rig & Hydrovac truck

REMARKS:

DATE: April 30, 2025

FILE NO. : TG0158

HOLE NO. : BH 5-25



TOPSOIL

GLACIAL TILL: Compact to very dense, red-brown clayey silt, with shale, gravel, cobbles and boulders

Dynamic cone penetration test commenced at 6.20 m depth

End of Borehole

Practical refusal to DCPT at 6.30 m depth

(GWL at 5.45 m depth - May 8, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
G 1			12
SS 2	100	6-7-8-13 15	22
SS 3	98	11-22-50-/ 72/0.28	10
SS 4	100	26-50-/-/ 50/0.08	6
SS 5	98	50-/-/-/ 50/0.13	9
SS 6	99	13-20-50-/ 70/0.25	7
SS 7	95	50-/-/-/ 50/0.08	4
SS 8	98	50-/-/-/ 50/0.13	4
SS 9	100	50-/-/-/ 50/0.13	5

P:/Autocad Drawings/Test Hole Data Files/TG01xx/TG0158/data.sqlite 2025-05-09, 10:41 Paterson_Template JM

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)

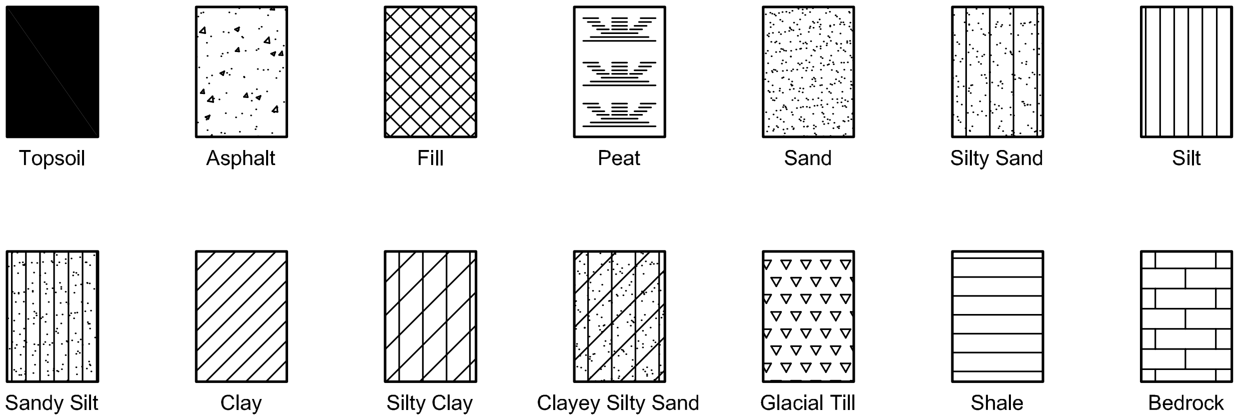
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

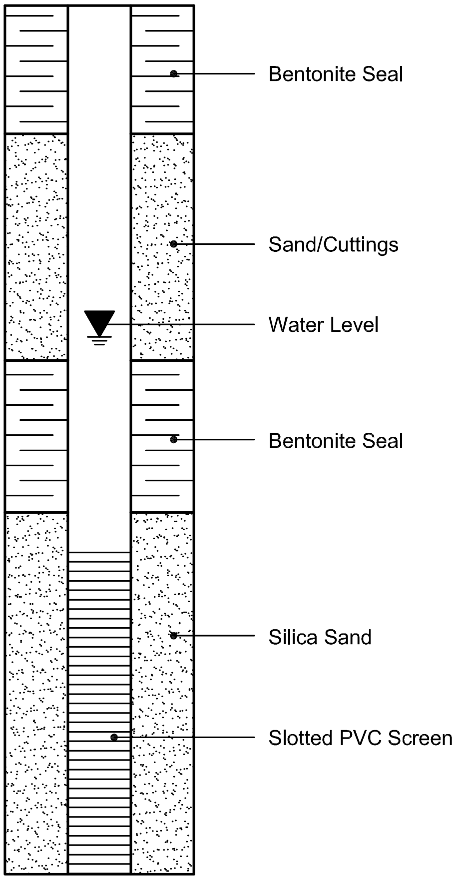
SYMBOLS AND TERMS (continued)

STRATA PLOT

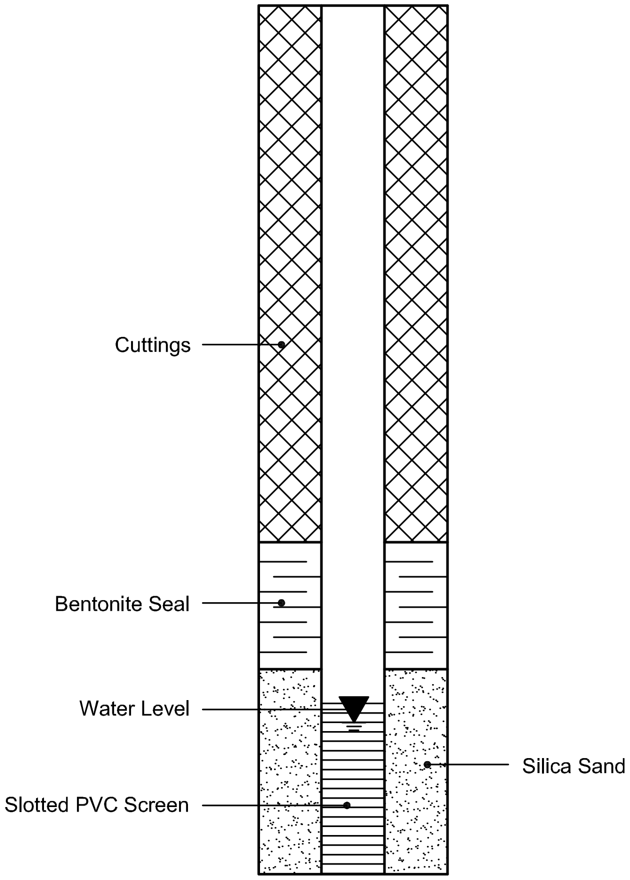


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers (Mississauga)**

Client PO: 62999

Report Date: 08-May-2025

Order Date: 2-May-2025

Project Description: TG0158

	Client ID:	BH3-25 SS2	-	-	-
	Sample Date:	30-Apr-25 12:00	-	-	-
	Sample ID:	2518478-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	80.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.04	-	-	-
Resistivity	0.10 Ohm.m	29.3	-	-	-
Anions					
Chloride	5 ug/g dry	56	-	-	-
Sulphate	5 ug/g dry	169	-	-	-

APPENDIX 2

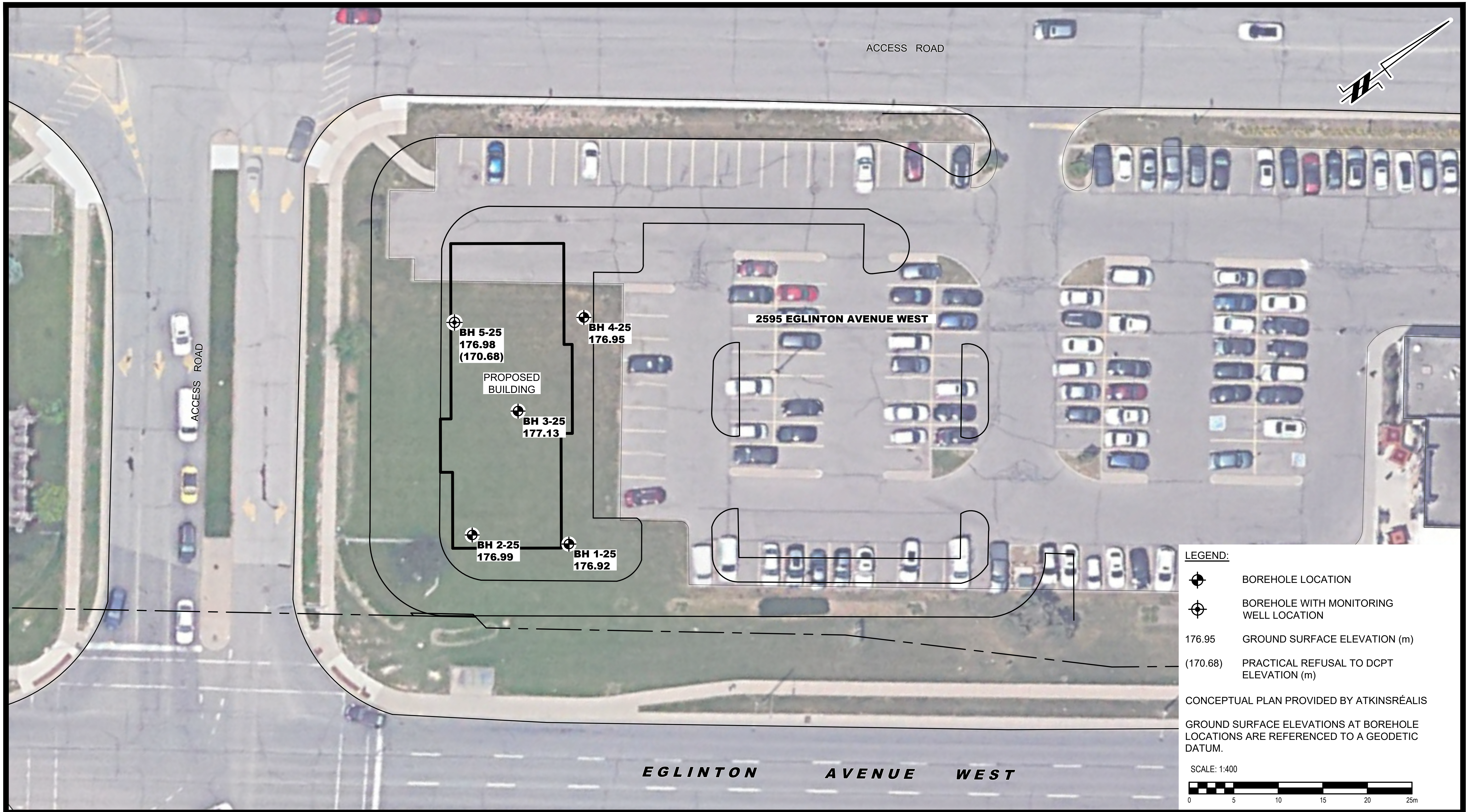
FIGURE 1 - KEY PLAN

DRAWING TG0158 - 1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON GROUP



LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
176.95	GROUND SURFACE ELEVATION (m)
(170.68)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY ATKINSRÉALIS

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

CHICK-FIL-A CANADA ULC
GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL BUILDING
2595 EGLINTON AVENUE WEST

MISSISSAUGA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400
Date: 05/2025
Drawn by: ZS
Report No.: TG0158-1
Checked by: BN
Dwg. No.: TG0158-1
Approved by: DP
Revision No.:

11x17

p:\autocad drawings\geotechnical\tg01xx\tg0158\tg0158-1-test hole location plan.dwg