Geotechnical Investigation

Proposed Multi-Storey Buildings

50 Bayswater Avenue & 1088 Somerset Street West
Ottawa, Ontario

Prepared for Manor Park Management

Report PG6565-1 Revision 1 dated February 25, 2025

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Manor Park Management to conduct a geotechnical investigation for the proposed multi-storey buildings to be located at 50 Bayswater Avenue and 1088 Somerset Street West in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

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

- ❑ Provide geotechnical recommendations pertaining to 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.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development at 1088 Somerset Street West will consist of a multi-storey, mixed-use structure (Building A) with one basement level for storage and mechanical space. The proposed development at 50 Bayswater Avenue will consist of a 16-storey, mixed-use structure (Building B) with 1 to 2 levels of underground parking.

Associated at-grade access lanes and parking areas, as well as amenity and landscaped areas, are anticipated immediately around the proposed buildings. It is also expected that the proposed buildings will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was conducted on February 13th and March 15th, 2023, and consisted of advancing 6 boreholes to a maximum depth of 6.75 m below the existing ground surface. All fieldwork was reviewed in the field by Paterson personnel under the direction of a senior engineer from the geotechnical division.

The boreholes at 1088 Somerset Street were advanced using a low-clearance track mounted drill rig operated by a two-person crew, while the boreholes in the existing underground parking garage at 50 Bayswater Avenue were completed using a portable drill rig operated by a two-person crew. The drilling procedure consisted of advancing to the required depths at the selected locations and sampling the overburden. The borehole locations were distributed in a manner to provide general coverage of the subject site. The locations of the boreholes are shown on Drawing PG6565-1- Test Hole Location Plan included in Appendix 2.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

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

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality. The subsurface conditions observed at the test pits were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets 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.

3.2 Field Survey

1088 Somerset Street West

The borehole locations and ground surface elevation at each borehole location were surveyed by Paterson using a handheld GPS unit. The borehole locations are referenced to a geodetic datum and were surveyed in the field by Paterson personnel. The borehole locations and ground surface elevation at the borehole locations are presented on Drawing PG6565-1- Test Hole Location Plan in Appendix 2.

50 Bayswater Avenue

The locations and ground surface elevations at the borehole locations were surveyed in the field by Paterson personnel. Ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), taking the elevation of the top spindle of a nearby existing fire hydrant. The location and ground surface elevation at each borehole location are shown on Drawing PG6565-1 - Test Hole Location Plan included in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were examined in our laboratory to review the results of the field logging. Soil samples will be stored for a period of 1 month after this report is completed, unless we are otherwise directed.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

50 Bayswater Avenue

The subject site is currently occupied by an underground parking structure for the existing residential tower. A single-storey commercial building is located over the underground parking structure near the corner of Bayswater Avenue and Somerset Street West. It is our understanding that the existing parking structure and commercial building will be demolished prior to construction of the proposed development.

The subject site is bordered by a residential tower and its associated parking structure to the south, Somerset Street West to the north, Bayswater Avenue to the east, and an access road to the west.

1088 Somerset Street West

The subject site is currently occupied by a two-storey residential dwelling and an at-grade parking area. The subject site is at grade with the neighboring properties at approximate geodetic elevation 62 m.

The subject site is bordered by Somerset Street West to the north, residential dwellings to the south, commercial/residential buildings to the west, and an access road to the east.

4.2 Subsurface Profile

50 Bayswater Avenue

Generally, the subsurface profile encountered below the concrete floor slab of the existing underground parking garage consists of a thin layer of crushed stone, followed by glacial till. The glacial till generally consists of dense, brown silty sand to sandy silt with gravel, cobbles, boulders.

Limestone bedrock was encountered underlying the glacial till at boreholes BH 4-23 and BH 6-23 at approximate depths of 3.5 and 3.2 m, respectively, below the floor slab of the underground parking structure. These depths correspond to geodetic elevations of about 55.3 and 55.5 m. The upper 0.5 m of the bedrock was cored at each of these locations.

The surficial bedrock in borehole BH 4-23 was observed to have an RQD value of 35%, indicative of poor quality bedrock, while the surficial bedrock in borehole BH 6-23 was observed to have an RQD of 68%, indicative of fair quality bedrock.

1088 Somerset Street West

The subsurface profile encountered at the borehole locations generally consists of a fill layer overlying glacial till. The fill material consists of brown silty with gravel and crushed stone, extending to depths varying from about 0.70 to 2.2 m below the existing ground surface. The glacial till was observed to consist of dense to very dense, brown to grey silty sand with gravel.

Practical refusal to augering was encountered at approximate depths ranging between 5.3 to 6.7 m below the existing ground surface. Based on available geological mapping, the bedrock in the area of the subject site consists of interbedded shale and limestone of the Verulam formation.

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

4.3 Groundwater

Groundwater levels were measured on February 2 and March 24, 2023 at the installed monitoring well locations. The groundwater level measurements are presented in Table 1 below.

Table 1 – Summary of Groundwater Levels

Borehole Number	Observation Method	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
			Depth (m)	Elevation (m)	
BH 1-23	Monitoring Well	61.85	4.61	57.24	February 22, 2023
BH 2-23	Monitoring Well	61.75	4.49	57.26	February 22, 2023
BH 3-23	Monitoring Well	62.33	5.14	57.19	February 22, 2023
BH 4-23	Monitoring Well	58.78	1.53	57.25	March 24, 2023
BH 5-23	Monitoring Well	58.74	1.17	57.57	March 24, 2023
BH 6-23	Monitoring Well	58.73	1.53	57.20	March 24, 2023

The groundwater table can also be estimated based on moisture levels and color of the recovered soil samples. Based on these observations at the borehole locations, the long-term groundwater table is expected at approximate geodetic elevation 57 to 58 m. It should also be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is recommended that the proposed building at 1088 Somerset Street West be founded on conventional spreading footings bearing on the undisturbed, dense glacial till.

For the proposed building at 50 Bayswater Avenue, it is recommended to be founded on conventional spread footings bearing on the clean, surface sounded bedrock. If this proposed building ends up having 1 level of underground parking, then lean concrete trenches will likely be required in order to extend the footing support down to the bedrock.

If the proposed building at 50 Bayswater Avenue has 2 levels of underground parking, bedrock removal will likely be required. All contractors should be prepared for bedrock removal at this site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and any fill containing significant amounts of deleterious or organic materials should be stripped from under the proposed building footprints. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Any soft areas should be removed and replaced in accordance with the following fill placement recommendations.

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

Bedrock Removal

If bedrock removal is required, it is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock for the underground parking level. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm per second (mm/s) during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

Vibration Considerations

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

The following construction equipment could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of a temporary shoring system with soldier piles or sheet piling will require these pieces of equipment. Vibrations, caused by blasting or construction operations, could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

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

These guidelines are for current construction standards. Considering there are several sensitive buildings in close proximity to the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, therefore, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed buildings.

Fill Placement

Fill used for grading beneath the building footprints, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It 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 building area should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Lean Concrete Filled Trenches

Where the proposed building footings are designed to bear on bedrock which is encountered below the underside of footing (USF) elevation, vertical trenches will need to be excavated to the clean, surface sounded bedrock, and backfilled with lean concrete to the founding elevation (minimum 17 MPa 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying clean, surface sounded bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on the undisturbed, dense glacial till bearing surface 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 was applied to the bearing resistance at ULS.

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.

The above-noted bearing resistance values at SLS for undisturbed, dense glacial till bearing surfaces will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Footings placed on clean, surface sounded bedrock, or on lean concrete which is placed directly over the clean, surface sounded bedrock, can be designed using a bearing resistance value of **2,500 kPa** at ULS, incorporating a geotechnical resistance factor of 0.5.

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

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a glacial till above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher bearing capacity as the bearing medium soil.

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

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the foundations considered at this site. If the proposed footings are within 3 m of the bedrock surface, and a higher seismic site class is required (Class A or B), a site specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed buildings, as presented in Table 4.1.8.4.A of the Ontario Building Code 2012.

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

5.5 Basement Slab

With the removal of all fill containing deleterious material within the footprints of the proposed buildings, the native soil surface will be considered an acceptable subgrade on which to commence backfilling for floor slab construction. Any soft areas should be removed and backfilled with appropriate backfill material such as OPSS Granular B Type II, with a maximum particle size of 50 mm.

For the proposed building at 1088 Somerset Street West, it is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone crushed stone.

For the proposed building at 50 Bayswater Avenue, it is understood that the underground level(s) will be mostly parking, and the recommended pavement structures noted in Section 5.8 will be applicable.

In consideration of the anticipated groundwater conditions, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the granular fill layer under the lowest level floor slab of each building. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed structures. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3.

Where undrained conditions are anticipated, the applicable effective unit weight of the retained soil can be designed with 13 kN/m^3. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight. The total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component (ΔP_{AE}).

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Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $Ko \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

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

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:
$a_c = (1.45\text{-}a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

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

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

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

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

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in bedded sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. Both modes of failure have to be examined, as described in the following section. A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

Anchors can be of the “passive” or the “prestressed/post-tensioned” type, depending on whether the anchor tendon is provided with prestress load or not prior to being put into service.

Regardless of whether an anchor is of the passive or the prestressed type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length.

As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a shallower cone, and therefore less geotechnical resistance, than one where the bonded length was just the bottom part of the overall anchor.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m.

Generally, the UCS of limestone ranges between about 60 and 90 MPa, which is stronger than most routine grouts. A factored at ULS tensile grout to rock bond resistance value of **1.0 MPa**, incorporating a resistance factor of 0.4, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system.

Based on bedrock information provided from the geotechnical investigation report, and assuming that the bond zone will be in competent bedrock located 1 m below the bedrock surface, a Rock Mass Rating (RMR) of 65 was assigned to the bedrock, and Hoek and Brown parameters (m and s) were taken as 0.575 and 0.00293, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2 below:

Table 2 - Parameters Used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone	60 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75- and 125-mm diameter hole are provided in Table 3 on the next page.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2.0	1.0	3.0	450
	3.0	1.5	4.5	700
	4.4	1.8	6.2	1,000
125	1.6	1.4	3.0	625
	2.5	1.5	4.0	980
	3.6	2.0	5.6	1,400

Other Considerations

The anchor drill holes should be a maximum of 1.5 to 2 times the rock anchor diameter, inspected by Paterson personnel and should be flushed clean prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor hole. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction and reviewed at the time of testing by Paterson field personnel. More information on proof testing can be provided upon request.

5.8 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the lowest underground parking level of the proposed building at 50 Baywater Avenue consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 4 below:

Table 4 - Recommended Rigid Pavement Structure – Underground Parking Level

Thickness (mm)	Material Description
150	**Exposure Class C2** – **32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE – Existing imported fill, or OPSS Granular B Type **I** or **II** material placed over bedrock.	

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

The flexible pavement structure presented in Table 5 below should be used for at grade access lanes and heavy loading parking areas.

Table 5 - Recommended Asphalt Pavement Structure – Access Lanes and Heavy Loading Parking Area

Thickness (mm)	Material Description
40	**Wear Course** - Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - OPSS Granular B Type II overlying the concrete podium deck	

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A composite drainage board (such as Delta Drain 6000 or approved equivalent) should be installed on the exterior portion of all foundation walls, and should extend from the top of footing elevation, up to 300 mm below finished grades. The composite drainage board should be hydraulically connected to 150 mm diameter sleeves cast in the foundation wall, just above the foundation wall/footing interface, to allow for the infiltration of water to flow to an interior perimeter drainage pipe. The drainage sleeves should be spaced at 3 m center-to-center along the perimeter foundation wall of each building.

The interior perimeter drainage pipe should consist of a 100 or 150 mm diameter, perforated and corrugated pipe which has a positive outlet, such as a gravity connection to the building's sump pit.

If the proposed building at 50 Bayswater Avenue has 2 levels of underground parking, then it is recommended to install a waterproofing membrane (such as Tremco Paraseal, or approved equivalent) as the outermost layer from the face of the foundation wall, and over the composite drainage board, recommended above. The waterproofing membrane is recommended to extend between the USF elevation, and up to 4 m below finished grades.

Underslab Drainage System

An underslab drainage system is recommended to control water infiltration below the lowest level floor slab for each building. For preliminary design purposes, it is recommended that 150 mm perforated pipes be placed at approximate 6 m centres underlying the lowest level floor slab. The spacing of the underslab drainage system should be confirmed by the geotechnical consultant at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls, where required, should consist of free-draining, non-frost susceptible granular materials, such as clean sand or OPSS Granular B Type I granular material.

6.2 Protection of Footings Against Frost Action

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 alone, or a combination of soil cover in conjunction with foundation insulation, should be provided in this regard.

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

However, the footings are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden should either be cut back at acceptable slopes or should be retained by a temporary shoring system from the start of the excavation until the structure is backfilled. Based on the proximity of the proposed buildings to the property lines, it is anticipated that temporary shoring will be required. Underpinning of foundations of the existing residential tower at 50 Bayswater Avenue may also be required, dependent on the depths of the proposed building foundations.

Unsupported Excavations

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

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

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

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

Temporary Shoring

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

Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system design.

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

The temporary system could consist of a soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be added to the earth pressures described below. These systems could be cantilevered, anchored, or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability.

The earth pressures acting on the shoring system may be calculated using the parameters provided in Table 6 below.

Table 6 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight(γ’), kN/m^3	13

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

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

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

Underpinning

Should the proposed building at 50 Bayswater Avenue have 2 underground parking levels, then underpinning of the existing residential tower foundations, adjacent to the proposed building, would likely be required. Further information can be provided once the final excavation depths have been determined.

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. However, when the bedding is located within bedrock subgrade, a minimum of 300 mm of OPSS Granular A should be placed for bedding for sewer or water pipes. 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.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations through the overburden materials and bedrock should be low to moderate and 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) 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.

Impact on Neighbouring Properties

Based on the depth of the dense glacial till encountered at the subject site, it is likely that the surrounding structures are founded on the dense glacial till. This material is not subject to settlement as a result of dewatering. Further, a waterproofing membrane will be installed for portions of the proposed buildings extending below the groundwater level. Therefore, temporary dewatering which may occur during the construction period will not impact neighbouring structures.

6.6 Winter Construction

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

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

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

6.7 Corrosion Potential and Sulphate

The results of analytical testing from an adjacent site 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 an aggressive to very 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 of the geotechnical aspects of the excavation contractor's shoring design, prior to construction.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Field density tests to determine the level of compaction achieved.

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

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should 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 Manor Park Management 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
Feb. 25, 2025
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Deepak K Rajendran, E.I.T.

Scott S. Dennis, P.Eng.

Report Distribution:

- ❑ Manor Park Management (Digital copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
50 Bayswater Drive and 1088 Somerset Street W.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** February 10, 2023

FILE NO. PG6565

HOLE NO. BH 1-23

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	61.85
Asphaltic concrete	0.05	AU	1				
FILL: Brown silty sand with gravel and crushed stone		SS	2	75	50+	1	60.85
		SS	3	67	50+		
	2.21					2	59.85
GLACIAL TILL: Dense to very dense, brown silty sand, some gravel		SS	4	75	44		
						3	58.85
		SS	5	67	42		
		SS	6	67	50+	4	57.85
		SS	7	67	50+		
	5.28					5	56.85

End of Borehole

Practical refusal to augering at 5.28m depth.

(GWL @ 4.61m - Feb. 22, 2013)

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
50 Bayswater Drive and 1088 Somerset Street W.
Ottawa, Ontario

DATUM Geodetic

REMARKS

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

FILE NO. PG6565

HOLE NO. BH 2-23

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	61.75
Asphaltic concrete 0.05						
	AU	1				
FILL: Brown silty sand with gravel and crushed stone	SS	2	58	49	1	60.75
	SS	3	25	41	2	59.75
2.21						
GLACIAL TILL: Dense to very dense, brown silty sand, some gravel	SS	4	75	50+		
					3	58.75
	SS	5	92	50+		
	SS	6	92	50+	4	57.75
	SS	7	67	50+		
					5	56.75
	SS	8	58	26		
					6	55.75
6.27	SS	9	40	50+		

End of Borehole

Practical refusal to augering at 6.27m depth.

(GWL @ 4.49m - Feb. 22, 2013)

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
50 Bayswater Drive and 1088 Somerset Street W.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE February 13, 2023

FILE NO. PG6565

HOLE NO. BH 3-23

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	62.33
Asphaltic concrete (0.08)						
FILL: Brown silty sand with topsoil, trace gravel (0.69)	AU	1				
GLACIAL TILL: Dense to very dense, brown silty sand, with gravel	SS	2	55	50+	1	61.33
	SS	3	60	50+	2	60.33
	SS	4	75	50+		
	SS	5	80	50+	3	59.33
- grey by 4.0m depth	SS	6	75	41	4	58.33
	SS	7	50	37	5	57.33
	SS	8	33	27		
(6.73)	SS	9	93	50+	6	56.33

End of Borehole

Practical refusal to augering at 6.73m depth.

(GWL @ 5.14m - Feb. 22, 2013)

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
50 Bayswater Drive and 1088 Somerset Street W.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE March 15, 2023

FILE NO. PG6565

HOLE NO. BH 4-23

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	58.78
Concrete	0.23						
FILL: Crushed stone and gravel	0.41						
GLACIAL TILL: Brown silty sand to sandy silt with gravel, cobbles and boulders, trace clay	3.53	SS	1	100			
		SS	2	33		1	57.78
		SS	3	100			
		SS	4	0		2	56.78
		SS	5	58		3	55.78
		SS	6	58			
BEDROCK: Poor quality, grey limestone	4.11	RC	1	100	35	4	54.78
End of Borehole							

(GWL @ 1.53m - March 24, 2023)

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
50 Bayswater Drive and 1088 Somerset Street W.
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG6565**

REMARKS

HOLE NO. **BH 5-23**

BORINGS BY CME-55 Low Clearance Drill **DATE** March 16, 2023

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Monitoring Well Construction
GROUND SURFACE						0	58.74		
Concrete 0.18									
FILL: Gravel and crushed stone 0.36									
		SS	1	42					
		SS	2	0		1	57.74		
GLACIAL TILL: Brown silty sand to sand silt with gravel and cobbles									
		SS	3	25		2	56.74		
- grey by 1.8m depth									
		SS	4	33					
		SS	5	33		3	55.74		
3.43									
End of Borehole									
(GWL @ 1.17m - March 24, 2023)									

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

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
50 Bayswater Drive and 1088 Somerset Street W.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE March 16, 2023

FILE NO. PG6565

HOLE NO. BH 6-23

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	58.73
Concrete	0.13						
FILL: Gravel and crushed stone	0.30						
GLACIAL TILL: Grey silty sand to sandy silt with gravel, cobbles and boulders		SS	1	50			
		SS	2	50		1	57.73
		SS	3	42		2	56.73
- grey by 2.4m depth		SS	4	8			
	3.22	SS	5	9		3	55.73
BEDROCK: Fair quality, grey limestone	3.71	RC	1	100	68		
End of Borehole							
(GWL @ 1.53m - March 24, 2023)							

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

○ Water Content %

20 40 60 80

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

Monitoring Well Construction

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.

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

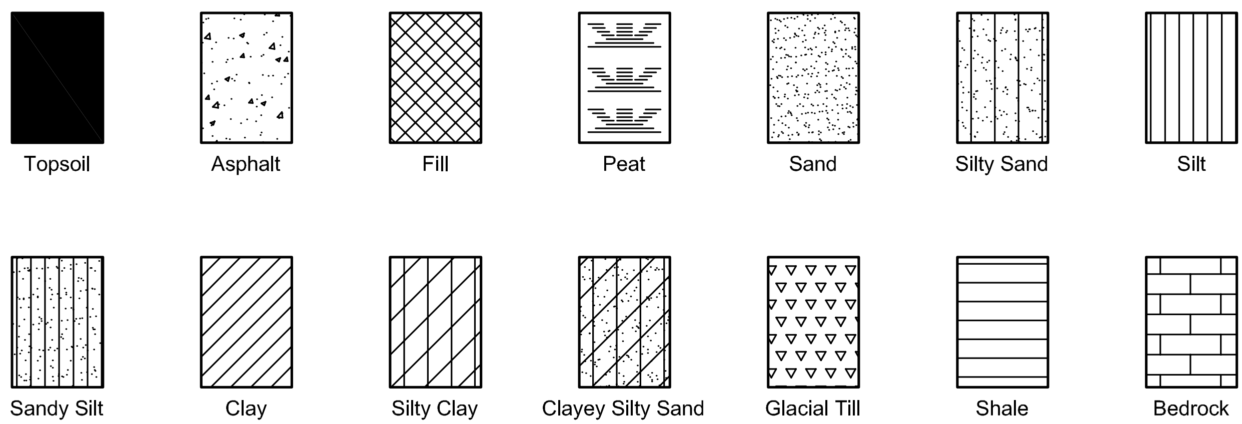
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

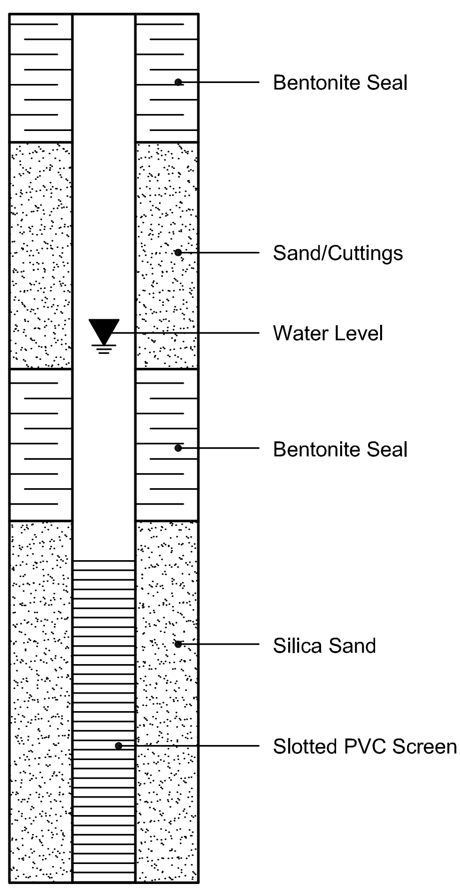
SYMBOLS AND TERMS (continued)

STRATA PLOT

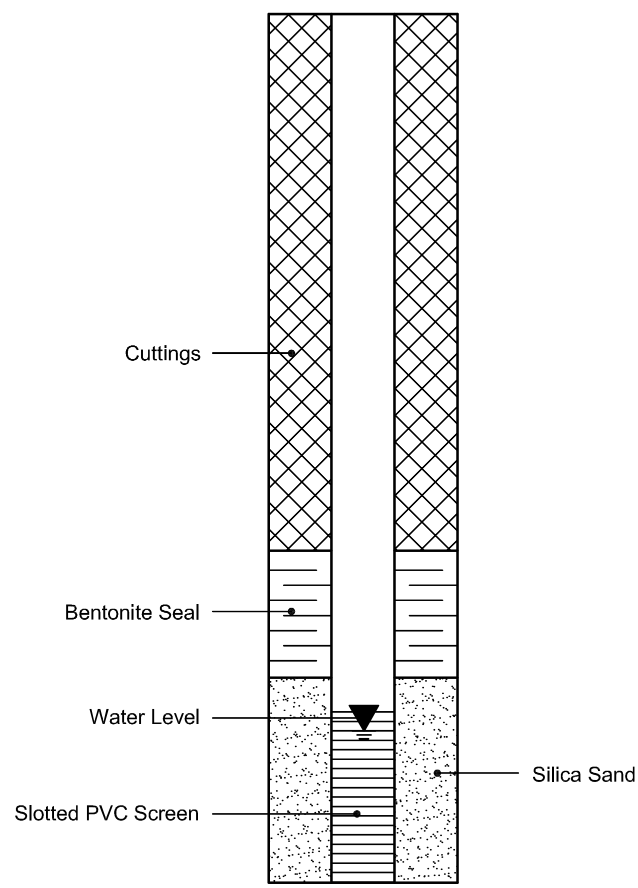


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: **57085**

Report Date: 30-Mar-2023

Order Date: 24-Mar-2023

Project Description: PG6565

	Client ID:	BH2-23 SS3	-	-	-
	Sample Date:	23-Mar-23 09:00	-	-	-
	Sample ID:	2312553-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.87	-	-	-
Resistivity	0.1 Ohm.m	8.1	-	-	-
Anions					
Chloride	10 ug/g dry	561	-	-	-
Sulphate	10 ug/g dry	353	-	-	-

APPENDIX 2

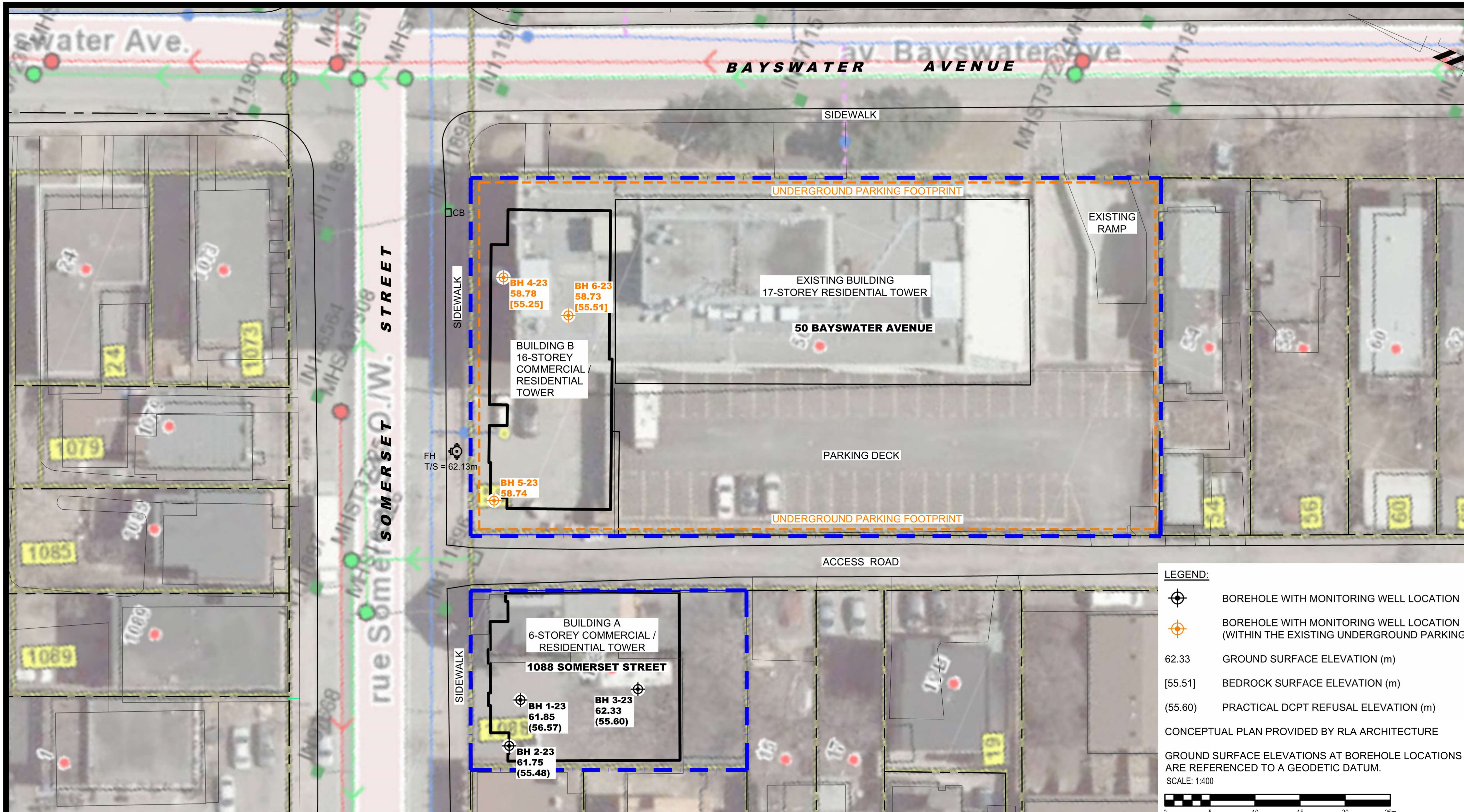
FIGURE 1 – KEY PLAN

DRAWING PG6565-1 – TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON GROUP



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NO.	REVISIONS	DATE	INITIAL

MANOR PARK MANAGEMENT
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT - MANOR PARK MANAGMENT
50 BAYSWATER AVENUE & 1088 SOMERSET STREET
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 02/2025
Drawn by: YA	Report No.: PG6565-1
Checked by: PB	Dwg. No.: PG6565-1
Approved by: SD	Revision No.: 1

p:\autocad drawings\geotechnical\pg65xx\pg6565\pg6565-1-test hole location plan.dwg

11x17