PATERSON GROUP

Geotechnical Investigation

Proposed Residential Development

1360 Ogilvie Road
Ottawa, Ontario

Prepared for Better Living Residential CO-OP

Report PG6694– 1 dated June 27, 2023

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Better Living Residential CO-OP to conduct a geotechnical investigation for the proposed residential development to be located at 1360 Ogilvie Road in Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

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

- ❑ Provide geotechnical recommendations pertaining to 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 property was not part of the scope of work of this present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on our review of available drawings, it is anticipated the proposed residential development will consist of a four-storey building with one underground parking level. The existing building and foundation located on the north of the site are expected to be demolished as part of the development.

Furthermore, reinstatement of the existing landscape area, parking space and access lane is expected. It is expected that the site will be serviced by existing storm, sanitary and water municipal services.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on June 2, 2023, and consisted of a total of four (4) boreholes sampled to a maximum depth of 6.2 m below ground surface throughout the subject site.

The test hole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground utilities, site features and the proposed development. The test hole locations are shown on Drawing PG6694-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were recovered 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) sample. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags.

All samples were transported to our laboratory for further examination and classification. The depths at which the auge and split spoon samples were recovered from the boreholes are shown as AU and SS respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

Undrained shear strength testing was carried out at regular depth intervals in cohesive soils.

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

Flexible standpipe piezometers were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented 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 ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS referenced to a geodetic datum. The locations of the test holes, and the ground surface elevation at each test hole location, are presented on Drawing PG6694-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were collected from the subject site during the investigation and were visually examined in our laboratory to review the results of the field logging. All samples were submitted for moisture content testing. The test results are included on the Soil Profile and Test Data sheets presented in Appendix 1.

All samples will be stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless otherwise directed.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an existing residential building which has been partially demolished to its foundation due to fire damage. An existing parking area and access lanes are located on the east side and west side of the demolished building. Furthermore, multiple existing multi-storey residential buildings are located on the east, west and south borders of the site.

The subject site is bordered to the north by Ogilvie Road, to the east by multi-storey residential buildings, to the south by residential dwellings and to the west by undeveloped land. The ground surface in proximity to the demolished building is generally flat at an approximate geodetic elevation of 72.3 to 72.7 m. The ground surface across the site slopes down from the west to the east.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile was observed to consist of asphaltic concrete overlying fill consisting of silty sand with crushed stone, gravel and varying amounts of clay and cobbles. The fill layer was noted to extend to depths ranging from 2.29 m to 3.66 m.

The fill layer in BH 1-23 to BH 3-23 was underlain by a layer of peat with topsoil to depths ranging from 3.05 m to 3.73 m. The peat with topsoil layer was further underlain by a glacial till deposit consisting of silty clay and silty sand with gravel to the maximum depth of sampling in BH 1-23 and BH 3-23. A thin layer of silty clay was noted in BH 2-23 below the peat with topsoil layer. The weathered shale bedrock layer was noted below the glacial till in BH 2-23 to the maximum depth of sampling.

Fill was noted to extend to the maximum depth of sampling in BH 4-23 consisting of varying amounts of silt, sand, clay, gravel, and shale.

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

Bedrock

Based on geological mapping and the results of the field investigation, the overburden drift thickness ranges between 5.3 to greater than 6.2 m and is underlain by shale of the Billings Formation.

4.3 Groundwater

Groundwater level readings were measured on June 8, 2023 and are presented in Table 1 below, and on the Soil Profile and Test Data sheets in Appendix 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-23	72.30	3.90	68.4	June 8, 2023
BH 2-23*	72.51	3.93	68.58	
BH 3-23	72.54	4.05	68.49	
BH 4-23	72.66	dry	dry	

Note:
- The ground surface elevations are referenced to a geodetic datum.
- * Borehole with groundwater monitoring well

It should be noted that groundwater levels can be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed color, moisture levels and consistency of the recovered soil samples.

Based on these observations, the long-term groundwater level is anticipated to be at a depth ranging between 3.9 m to 4.1 m. However, groundwater levels are subject to seasonal fluctuations and could vary during the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

Foundation Design Considerations

From a geotechnical perspective, the subject site is suitable for the proposed development. It is recommended that the proposed buildings be founded on conventional spread footings bearing on the undisturbed compact glacial till and/or approved engineered fill. Due to the layer of fill and organic soil encountered in the boreholes, sub-excavation will be required to reach a competent undisturbed native bearing surface.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant organic materials, should be stripped from under any buildings and other settlement sensitive structures. The existing fill material, where free of organic materials, should be reviewed by Paterson personnel at the time of construction to determine if the existing fill can be left in place below paved areas and below the slab granular fill layers.

The layer of peat and topsoil noted below the fill layer is to be removed from under the entire proposed building area to the extents of the lateral support zone of the footings, which may result in sub-excavation below the underside of footing elevation. It should be noted that the peat and topsoil is also to be removed from under the basement floor slab to prevent settlement of the slab.

Fill Placement

Fill placed for grading beneath the building areas and where sub-excavation was required to remove the peat and topsoil layer 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. The fill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill 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 thin lifts to a minimum density of 98% of their respective SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geo-composite drainage membrane connected to a perimeter drainage system.

5.3 Foundation Design

Conventional Spread Footings

Footings placed on an undisturbed compact/stiff glacial till, or on engineered fill, which is placed and compacted directly over this strata, can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical resistance factor of 0.5 was applies to the bearing resistance value at ULS.

An undisturbed soil bearing surface consists of a surface 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 bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support. Adequate lateral support is provided to a soil bearing medium above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the foundations considered as defined in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2020. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the OBC 2020 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil, peat and deleterious materials within the footprint of the proposed building, a soil subgrade approved by Paterson personnel at the time of construction, is considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab construction.

The recommended pavement structures noted in Subsection 5.7 will be applicable where the basement level underlying foundation support consists of conventional spread footings.

For structures with slab-on-grade construction, the upper 300 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprints of the proposed buildings should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of its SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

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

5.6 Basement Wall

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

The total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component ($\triangle P_{AE}$).

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32 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 Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 2. The flexible pavement structure presented in Table 3 and Table 4 should be used for driveways and car only parking areas and at grade access lanes and heavy loading parking areas.

Table 2 - Recommended Rigid Pavement Structure - Lower Parking Level	
Thickness (mm)	**Material Description**
150	**Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** – OPSS Granular A Crushed Stone
SUBGRADE – Compact to dense glacial till, or OPSS Granular A or OPSS Granulart B Type II granular fill material placed over in situ soil.	

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 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

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

Table 4 - Recommended Pavement Structure – Heavy-Truck Traffic, Loading Areas and Access Lanes	
Thickness (mm)	**Material Description**
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil, bedrock or fill.	

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 material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Consideration should be given to installing subdrains during the pavement construction. The invert of the subdrain pipe is recommended to be located a minimum depth of 300 mm below the pavement structure subgrade and located centrally along the roadway alignment. The subdrain pipe is recommended to consist of a minimum 150 mm diameter corrugated and perforated plastic pipe surrounded by a minimum of 150 mm of 10 mm clear crushed stone on all of its sides. The clear stone layer is recommended to be wrapped by a geotextile layer. The drains should be connected to a positive outlet. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended to be provided for the proposed building. The system should consist of a 150 mm diameter perforated and corrugated plastic pipe, surrounded on all-sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer or sump pit provided in the lowest basement level of the structure.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials.

The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type II granular material, should otherwise be used for this purpose.

Foundation backfill material should be compacted in maximum 30 mm thick loose lifts and with suitably sized vibratory compaction equipment (smooth-drum roller for crushed stone fill, sheepsfoot roller for soil fill).

Interior Perimeter and Underfloor Drainage

An interior underfloor drainage system is recommended to redirect water from the building's foundation drainage system to the buildings sump pit(s) if it will not discharge to an exterior catch basin structure. The interior perimeter and underfloor drainage pipe should consist of a 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves. The spacing of the underfloor drainage system should be confirmed by Paterson once the foundation layout and sump system location has been finalized.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

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. Generally, a minimum of 1.5 m thick soil cover (or an equivalent combination of soil cover and foundation insulation) should be provided in this regard.

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

6.3 Excavation Side Slopes

The side slopes of the shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by temporary shoring systems from the start of the excavation until the structure is backfilled.

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m, should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below ground water level. The subsoil at this site appeared to be mainly a Type 2 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 may be required for the overburden soil to complete the required excavations 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. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored, or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated using the parameters provided in Table 5.

Table 5 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System	
Parameter	**Value**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ'), kN/m^3	13

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

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

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

6.4 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A crushed stone should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material’s standard Proctor maximum dry density.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize potential differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% of the material's SPMDD. All cobbles larger than 200 mm in the longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

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

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations through the overburden materials 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 the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

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

6.6 Winter Construction

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

In the event of construction during below zero temperatures, the founding 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 in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at the subject site, whereas the resistivity is indicative of an aggressive to very aggressive corrosive environment.

7.0 Recommendations

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

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

For the foundation design data provided herein to be applicable, a material testing and observation services program is required to be completed. The following aspects be performed by Paterson:

- ❑ 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.
- ❑ Observation of the placement of the foundation insulation, if applicable.
- ❑ 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 the construction has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program 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 herein 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 Better Living Residential CO-OP, 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.

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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
Prop. Residential Development - 1360 Ogilvie Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 2, 2023

FILE NO. PG6694

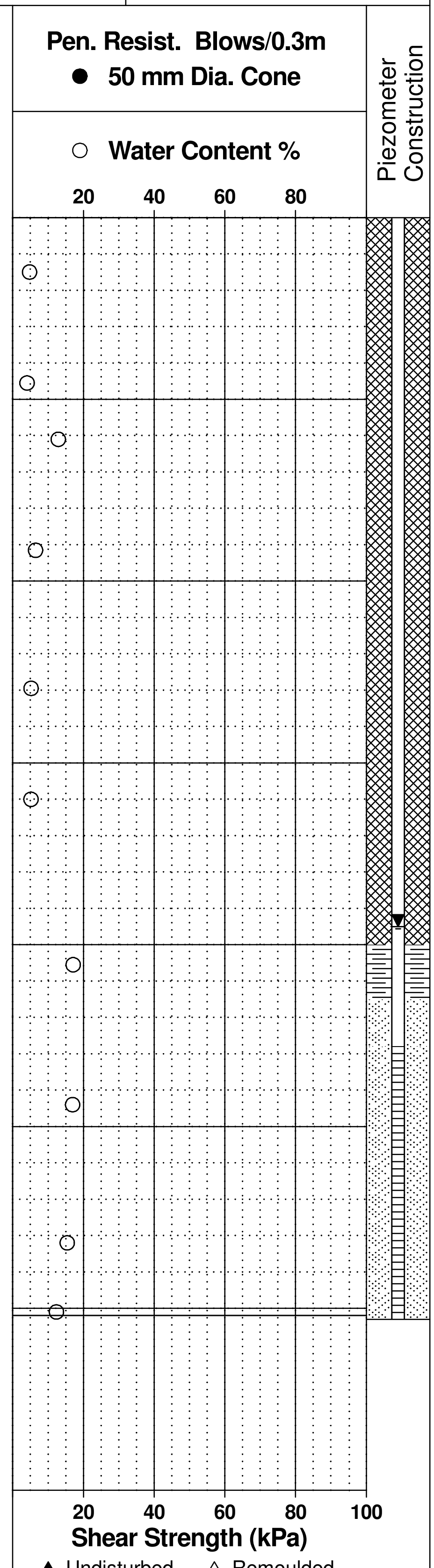
HOLE NO. BH 1-23

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
Asphaltic concrete	0.05				
FILL: Dark brown silty sand with crushed stone and gravel, occasional cobbles, trace clay	3.58	SS	1	50	19
		SS	2	71	12
		SS	3	42	16
		SS	4	38	16
		SS	5	62	21
PEAT with topsoil	3.73				
GLACIAL TILL: Stiff, dark grey silty clay, some sand, trace gravel, occasional cobbles	3.96	SS	6	58	12
GLACIAL TILL: Compact to loose, dark grey silty sand with gravel		SS	7	50	13
- trace clay by 6.0m depth	6.04	SS	8	38	4
		SS	9	100	50+
End of Borehole					

Practical refusal to augering at 6.09m depth.

(GWL @ 3.90m - June 8, 2023)

DEPTH (m)	ELEV. (m)
0	72.30
1	71.30
2	70.30
3	69.30
4	68.30
5	67.30
6	66.30



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 1360 Ogilvie Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** June 2, 2023

FILE NO. PG6694

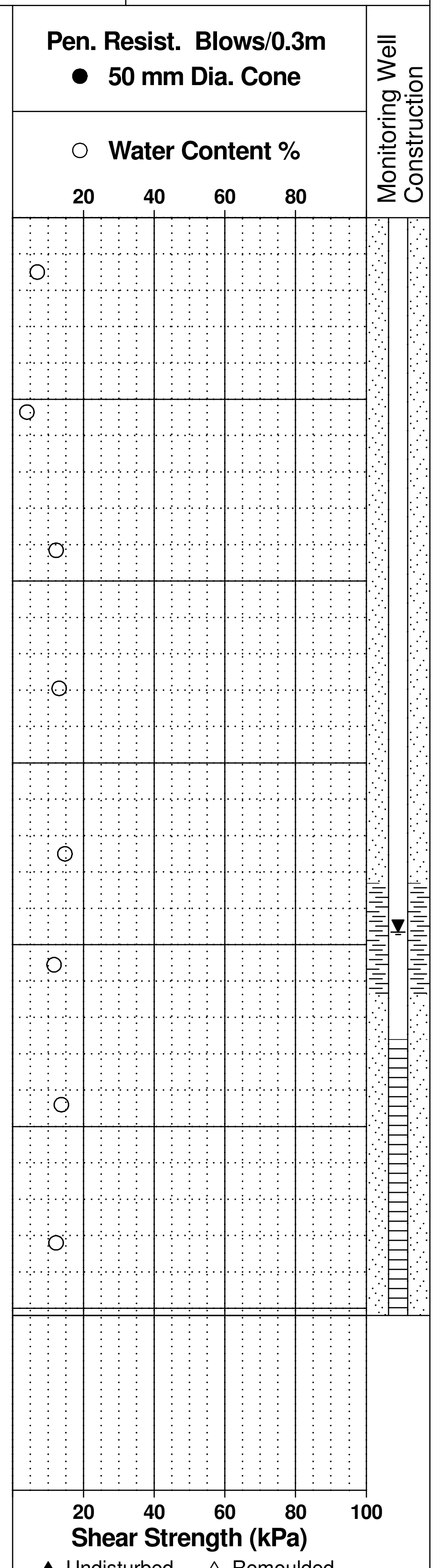
HOLE NO. BH 2-23

SOIL DESCRIPTION	Strata depth (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	72.51
Asphaltic concrete	0.03						
FILL: Dark brown silty sand with crushed stone and gravel, occasional cobbles, trace clay		SS	1	75	29		
		SS	2	83	50+	1	71.51
		SS	3	75	27	2	70.51
	3.05	SS	4	71	21	3	69.51
PEAT with topsoil	3.35						
Very stiff, brown **SILTY CLAY,** some sand	3.50	SS	5	79	9		
GLACIAL TILL: Compact to dense, grey silty sand with gravel, occasional cobbles		SS	6	67	14	4	68.51
	5.33	SS	7	58	25	5	67.51
Weathered **BEDROCK**	6.04	SS	8	75	26	6	66.51

End of Borehole

Practical refusal to augering at 6.04m depth.

(GWL @ 3.93m - June 8, 2023)



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 1360 Ogilvie Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

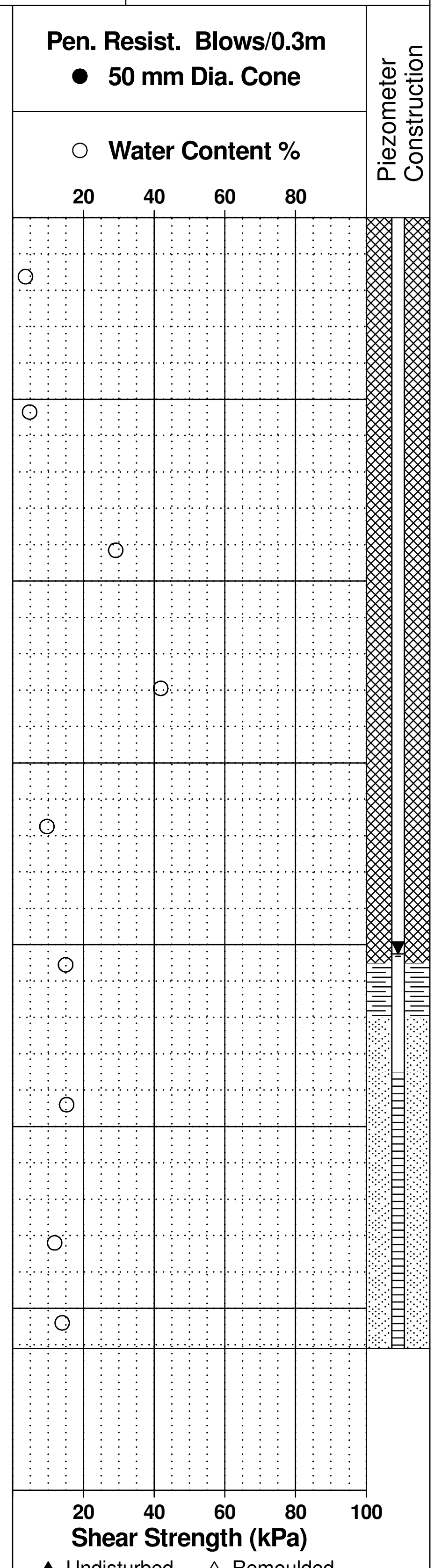
BORINGS BY CME-55 Low Clearance Drill

DATE June 2, 2023

FILE NO. PG6694

HOLE NO. BH 3-23

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface					0	72.54
Asphaltic concrete (0.05)						
FILL; Dark brown silty sand with crushed stone and gravel, occasional cobbles, trace clay	SS	1	62	29		
	SS	2	58	16	1	71.54
(1.45)						
FILL: Dark grey silty clay, some sand, trace gravel	SS	3	42	9	2	70.54
(2.29)						
PEAT with topsoil	SS	4	62	8		
(3.05)					3	69.54
GLACIAL TILL: Compact to very dense, dark brown silty sand with gravel, occasional cobbles	SS	5	75	15		
	SS	6	54	9	4	68.54
- dark grey by 4.6m depth	SS	7	83	36	5	67.54
	SS	8	58	50+		
	SS	9	100	50+	6	66.54
(6.22) End of Borehole						
(GWL @ 4.05m - June 8, 2023)						



patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - 1360 Ogilvie Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

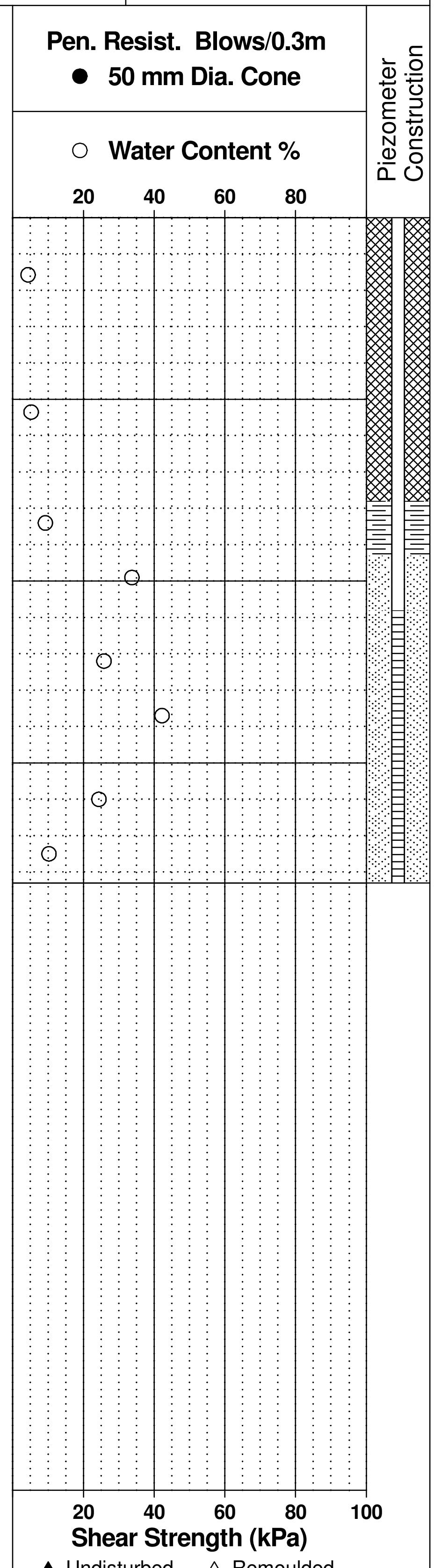
BORINGS BY CME-55 Low Clearance Drill **DATE** June 2, 2023

FILE NO. PG6694

HOLE NO. BH 4-23

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
Ground Surface					
Asphaltic concrete	0.03				
FILL: Dark brown silty sand with crushed stone and gravel, occasional cobbles, trace clay	1.83	SS	1	54	18
		SS	2	58	32
FILL: Brown silty clay, some sand	2.21	SS	3	67	38
FILL: Dark brown silty sand with topsoil	2.74	SS	4	79	9
TOPSOIL	2.90				
FILL: Dark brown silty sand with gravel, some shale	3.66	SS	5	88	14
End of Borehole					
(BH dry - June 8, 2023)					

DEPTH (m)	ELEV. (m)
0	72.66
1	71.66
2	70.66
3	69.66



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Retaining Wall - 1350 Ogilvie Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE June 10, 2021

FILE NO. PG5850

HOLE NO. BH 1-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.04				
FILL: Crushed stone, trace sand	0.60	AU	1		
FILL: Brown silty sand, trace gravel, crushed stone and topsoil	1.45	SS	2	17	8
FILL: Highly organic topsoil, some sand, gravel and shale fragments	2.97	SS	3	33	7
		SS	4	58	16
FILL: Brown silty sand with gravel, some crushed stone and shale fragments	4.42	SS	5	58	14
		SS	6	83	11
End of Borehole					
(GWL @ 2.94m - June 15, 2021)					

DEPTH (m)	ELEV. (m)
0	72.31
1	71.31
2	70.31
3	69.31
4	68.31

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

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

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

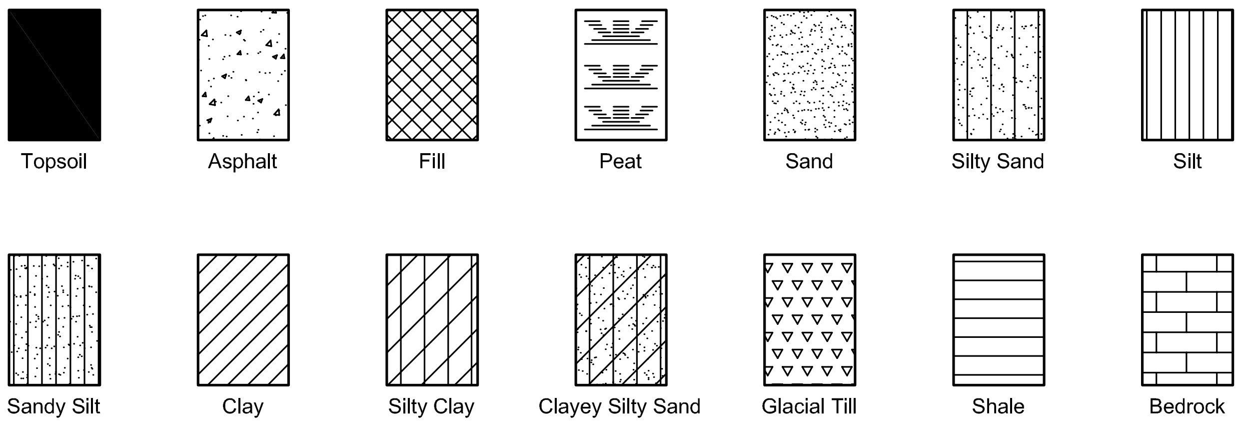
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

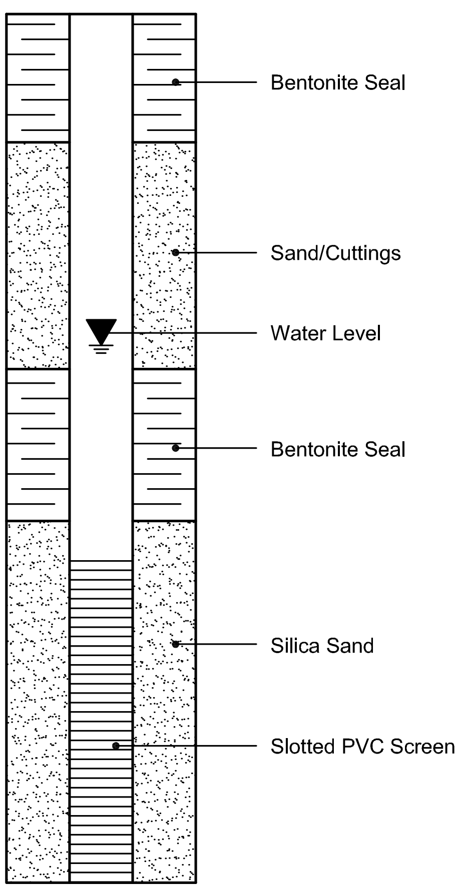
SYMBOLS AND TERMS (continued)

STRATA PLOT

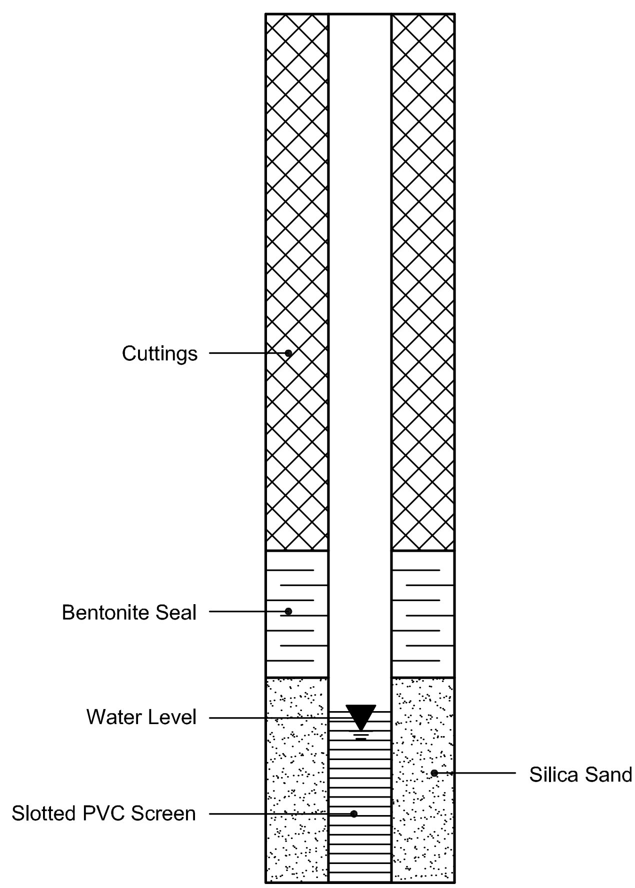


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 57724

Report Date: 20-Jun-2023

Order Date: 14-Jun-2023

Project Description: PG6694

	Client ID:	BH2-23-SS4	-	-	-
	Sample Date:	02-Jun-23 09:00	-	-	-
	Sample ID:	2324318-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.60	-	-	-
Resistivity	0.1 Ohm.m	6.6	-	-	-
Anions					
Chloride	10 ug/g dry	819	-	-	-
Sulphate	10 ug/g dry	507	-	-	-

APPENDIX 2

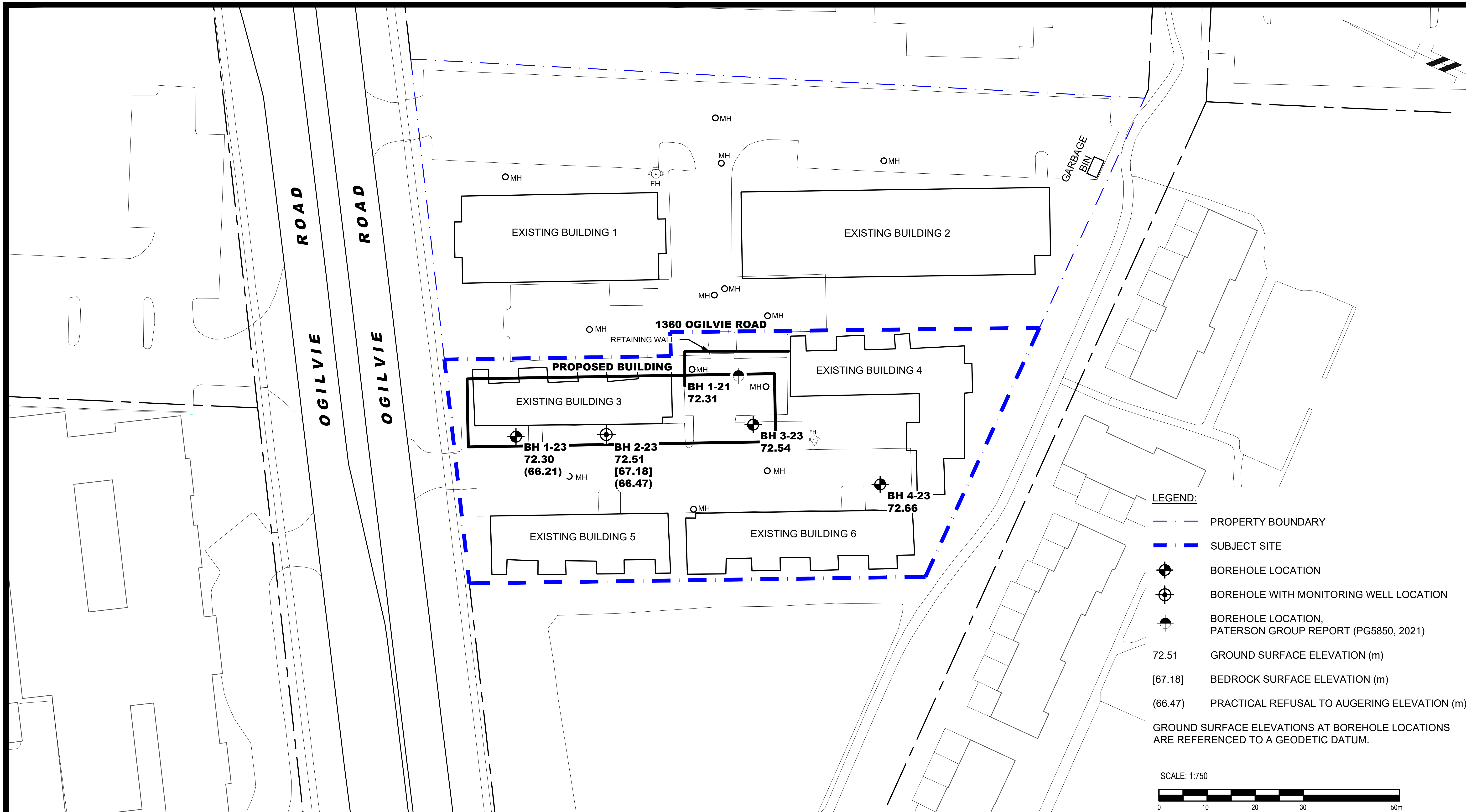
FIGURE 1 – KEY PLAN

DRAWING PG6694-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

PATERSON GROUP



PATERSON GROUP
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NO.	REVISIONS	DATE	INITIAL

BETTER LIVING RESIDENTIAL CO-OP

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
1360 OGILVIE ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 06/2023
Drawn by: GK	Report No.: PG6694-1
Checked by: NS	Dwg. No.: PG6694-1
Approved by: JV	Revision No.:

11x17

p:\autocad drawings\geotechnical\pg66xx\pg6694\pg6694-1 test hole location plan.dwg