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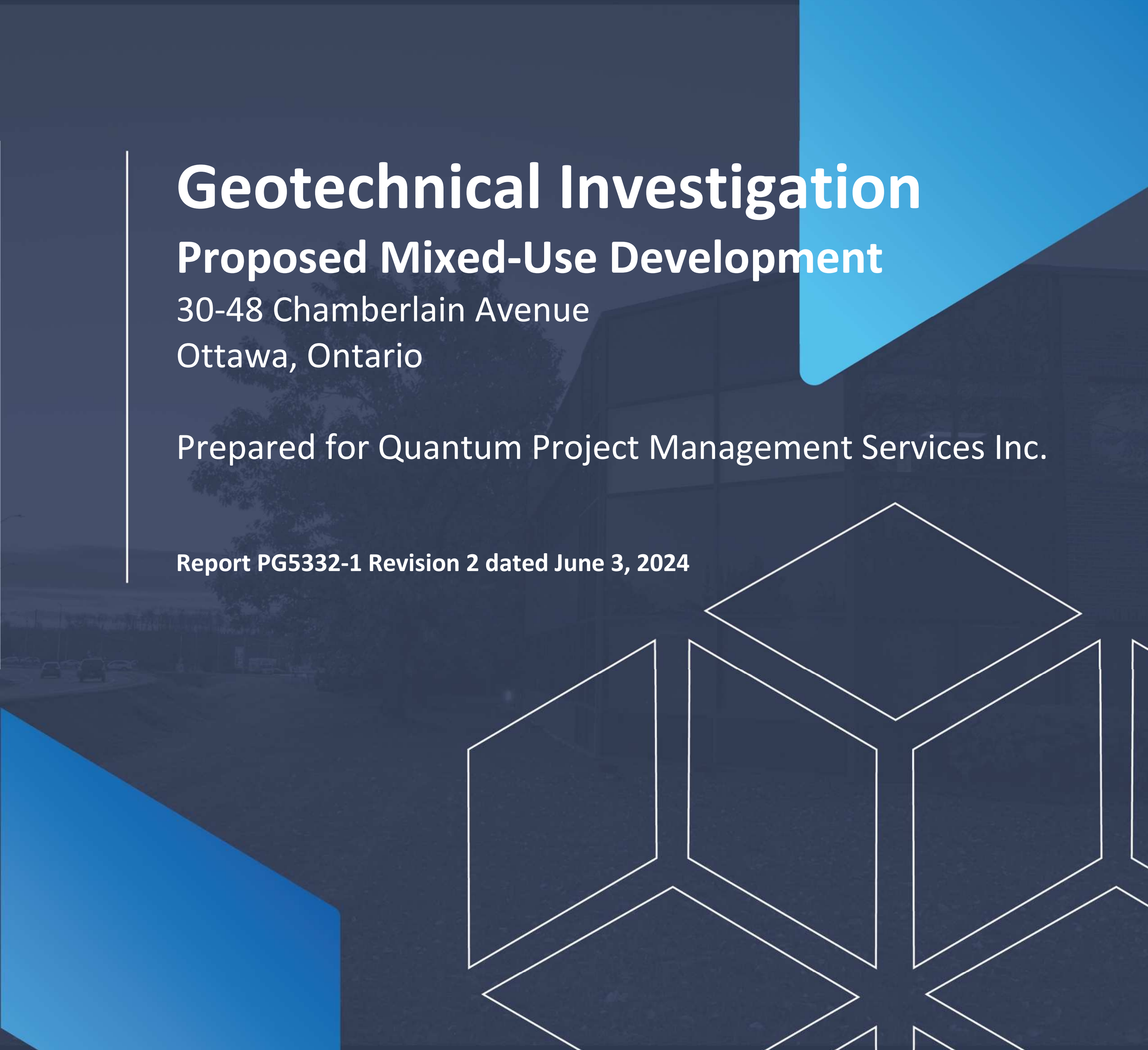

Geotechnical Investigation

Proposed Mixed-Use Development

30-48 Chamberlain Avenue
Ottawa, Ontario

Prepared for Quantum Project Management Services Inc.

Report PG5332-1 Revision 2 dated June 3, 2024

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Quantum Project Management Services Inc. to conduct a geotechnical investigation for the proposed mixed-use development to be located at 30-48 Chamberlain Avenue in the City of Ottawa, Ontario (refer to Figure 1 – Key Plan in Appendix 2 of this report for the general site location).

The objectives of the geotechnical investigation were to:

- ☐ Determine the subsurface soil and groundwater conditions based on test hole information completed within the subject site.

- ☐ 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. The report contains our findings and includes geotechnical recommendations pertaining to the design and construction of the proposed development as understood at the time of this report.

2.0 Proposed Development

Based on the current conceptual drawings, it is understood that the proposed development will consist of a high-rise building with a multi-storey podium structure which extends to the west and the east beyond the footprint of the high-rise. Further, it is understood that the first floor of the proposed podium structure will be used for commercial purposes while the remaining levels will be dedicated to residential use. The proposed building will also include 2 levels of underground parking which will occupy the majority of the subject site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on May 21 and May 22, 2020. At that time, 5 boreholes (BH 1 through BH 5) were advanced to a maximum depth of 14.7 m below the existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the test holes are shown on Drawing PG5332-1 – Test Hole Location Plan included in Appendix 2.

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

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. All samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the auger and split spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

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

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH 5. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm.

The number of blows required to drive the cone into the soil is recorded for each 300 mm increment. The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Groundwater monitoring wells were installed in BH 1, BH 2 and BH 3 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. A flexible polyethylene standpipe was installed within BH 4 and BH 5 to measure the stabilized groundwater levels subsequent to completion of the sampling program.

Sample Storage

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

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 with respect to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5332-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its concentration of sulphate and chloride along with its resistivity and pH. The laboratory test results are shown in Appendix 1 and are discussed in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The site is partially occupied by three existing buildings consisting of several two- to three storey commercial units. The remainder of the site is generally occupied by asphalt-paved access lanes and parking areas with some landscaping.

The site is bordered by Chamberlain Avenue to the north, an Embassy property followed by parking areas to the west, a paved parking area followed by the associated commercial building to the east and residential dwellings to the south. The existing ground surface across the site is relatively flat and at grade with adjacent properties and roadways.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of asphalt underlain by fill extending to an approximate depth of 0.6 to 2.3 m below the existing ground surface. The fill was generally observed to consist of a compact brown silty sand with crushed stone and occasional brick, metal, and plastic fragments.

A silty clay deposit was encountered underlying the fill. This deposit was observed to consist of a very stiff to stiff, brown silty clay, becoming a stiff grey silty clay below approximate depths of 4.6 to 4.9 m below the existing ground surface.

Underlying the silty clay deposit below approximate depths of 6 to 7.6 m, a layer of sandy silt to silty sand was encountered. The sandy silt to silty sand layer was further underlain by a glacial till deposit at approximate depths of 7 to 9.3 m below the existing ground surface. The glacial till deposit was observed to consist of a grey silty clay with sand, some gravel, cobbles and boulders.

Practical refusal to augering or the DCPT was encountered at depths ranging from 11.0 to 14.7 m below the existing ground surface.

Bedrock

Based on available geological mapping, the bedrock at the subject site consists of limestone with interbedded shale of the Verulam formation across the west half of the subject site, and transitions to shale of the Billings formation across the east half of the subject site with a drift thickness of 10 to 15 m.

4.3 Groundwater

Groundwater levels measured in the standpipes are summarized in Table 1.

Table 1 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH1	67.53	2.44	65.09	May 29, 2020
BH2	67.49	4.01	63.48	
BH3	67.77	4.43	63.34	
BH4	67.48	4.55	62.93	
BH5	67.57	Blocked	-	

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately **4 to 5 m** below ground surface within the silty clay layer. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

It should 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 considered suitable for the proposed development. Based on the subsurface conditions encountered in the test holes and the anticipated building loads, foundation support for the proposed high-rise structure is recommended to consist of a raft foundation bearing on the undisturbed, compact sandy silt to silty sand or the undisturbed glacial till deposit. End-bearing, deep foundations are also considered for the proposed building as a sufficient foundation system.

Further, it is expected that the portion of the podium beyond the footprint of the high- rise will be founded on conventional shallow footings or a raft foundation bearing on the undisturbed compact sandy silt to silty sand deposit or the undisturbed glacial till deposit.

Due to the presence of the silty clay deposit, a permissible grade raise restriction will be required for the proposed grading.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas and other settlement sensitive structures. Existing construction debris should be entirely removed from within the perimeter of all buildings.

Vibration Considerations

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

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations could be the cause or source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

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

It should be noted that these guidelines are for today’s construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre- and post-construction survey be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Fill Placement

Fill used for grading beneath the proposed building should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building and paved areas should be compacted to at least 98% of the material’s 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. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material’s SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

Protection of Subgrade

It is recommended that a minimum 50 to 75 mm thick lean concrete mud slab be placed on the undisturbed, compact sandy silt to silty sand or undisturbed glacial till subgrade shortly after the completion of the excavation in order to protect the subgrade from disturbance, and/or to provide a suitable working surface for the piling rig (if piles are used).

The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment. The final excavation to the bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the silty sand to potential disturbance due to drying.

Pressure Relief Chamber

Should a raft foundation be utilized, a pressure relief chamber is recommended to be installed along with collection pipes within excavated within the undisturbed, compact sandy silt to silty sand or undisturbed glacial till deposit. The collection pipe trenching should extend along the proposed building perimeter and lead to the pressure relief chamber.

It is suggested that the pressure relief chamber be incorporated in the lowest section of the basement level within a utility room in close proximity to the proposed sump pit(s). Figure 2 - Pressure Relief Chamber in Appendix 2 provides an example of the required pressure relief chamber. Once the pressure relief chamber and associated piping is installed, the proposed raft slab can be constructed. The purpose of the pressure relief chamber will be as follows:

- ☐ Manage any water infiltration along the founding surface during the excavation program.
- ☐ Manage the water infiltration during the pouring of the raft slab to prevent water flow in the fresh concrete.
- ☐ Manage water infiltration below the raft slab until sufficient load is applied to resist any potential hydrostatic uplift.
- ☐ Regulate the discharge valve to control water infiltration once the raft slab is in place and over the long term to manage the hydrostatic pressure to permit any repairs associated with any water infiltration.
- ☐ Once sufficient load is applied to the raft slab, the pressure relief valve will be fully closed to prevent any further dewatering.

Hydrostatic Pressure

With the fully closed valve within the pressure relief chamber and a perfectly watertight foundation, it is expected that a maximum hydrostatic pressure of **20 kPa** for two basement levels, will be developed over the long term and should be incorporated in the design of the raft foundation and the foundation walls.

5.3 Foundation Design

Spread Footing Foundations

Footings placed on an undisturbed, very stiff to stiff silty clay, compact sandy silt to silty sand or glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical factor of 0.5 was incorporated to the bearing resistance value at ULS.

Footings designed using the bearing resistance value at SLS will be subjected to potential post-construction total and differential settlement of 25 and 20 mm, respectively.

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.

Raft Foundation

Based on the anticipated loads, it is recommended that foundation support for the high-rise portion of the proposed building consist of a raft foundation bearing on the undisturbed, compact sandy silt to silty sand or undisturbed glacial till deposit. A raft foundation may also be required for the portion of the proposed building beyond the footprint of the high-rise.

For 2 levels of underground parking, it is anticipated that the excavation will extend to a depth such that the underside of the raft slab would be placed between geodetic elevations of 59.5 to 60.5 m. The contact pressure provided considers the stress relief associated with the soil removal required for 2 levels of underground parking.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load.

For 2 levels of underground parking, a bearing resistance value at SLS (contact pressure) of **240 kPa** will be considered acceptable for a raft supported on the undisturbed, compact sandy silt to silty sand or undisturbed glacial till. It should be noted that the weight of the raft slab and everything above must be included when designing with this value. The factored bearing resistance (contact pressure) at ULS can be taken as **360 kPa**. For this case, the modulus of subgrade reaction was calculated to be **10 MPa/m** for a contact pressure of **240 kPa**.

The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

Based on the following assumptions for the raft foundation, the proposed building can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a silty clay, sandy silt/silty sand, or glacial till bearing medium when a plane extending down and out from the bottom edge of the footing or raft at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as the soil. It should be noted that where the foundation extends below the groundwater level, the effective unit weight should be utilized for the saturated portion of the soil or fill.

Deep Foundation – High-rise Building

For support of the proposed high-rise buildings, and where the proposed raft bearing capacity is not sufficient to withstand the imposed structural loads, then consideration could be given to using concrete filled steel pipe piles driven to refusal on the bedrock surface.

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 2 on the next page. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	910	1090	10	29
245	11	1050	1260	10	35
245	13	1250	1500	10	41

As a minimum, the pipe piles should be equipped with a base plate having a thickness of at least 20 mm to reduce potential damage to the pile tip during driving.

Provision should be made for restriking all of the piles at least once, 48 hours after the initial driving, to confirm the design set and/or the permanence of the set, and to check for upward displacement due to driving adjacent piles.

It is recommended that a pile load test or dynamic monitoring and capacity testing be carried out at an early stage during the piling operations to verify the transferred energy from the piledriving equipment and determine the load carrying capacity of the piles. The recommended number of tests is dependent on the number of piles and pile sizes; as a guideline a minimum of 2 tests per pile size should be carried out. It is also recommended that the tested pile locations be spread out across the proposed building footprints.

The post construction settlement of structural elements which derive their support from piles bearing on bedrock should be negligible.

If piles are to be left exposed during winter months, some form of frost protection will be required to prevent frost adhesion and jacking of the piles. Further guidelines can be provided on these measures at the time of construction, if required.

Permissible Grade Raise Recommendations

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **2.0 m** is recommended for grading at the subject site.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D**, however, a site specific shear wave velocity test could be completed to determine if a seismic Site Class C is applicable for foundation design of the proposed building with two underground parking levels. The 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

Where a raft slab is utilized, a granular layer of OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

For any portions of the proposed building founded on footings or piles, it is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

A sub-slab drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided under the lowest level floor slab. The spacing of the sub-slab drainage pipes can be determined at the time of construction to confirm groundwater infiltration levels, if any. This is discussed further in Subsection 6.1.

5.6 Basement wall

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

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

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 (Po) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP) can be calculated using $0.375 \cdot a \cdot \gamma \cdot H^2/g$ 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 0.32g 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

Car only parking areas, heavy truck parking areas and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 3 and 4.

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 Access Lanes, Ramp and Heavy Truck Parking Areas

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 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 I or Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable compaction equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage and Waterproofing

For the proposed underground parking levels, it is expected that the building foundation walls will be placed in close proximity to the site boundaries. Therefore, it is recommended that the foundation walls be blind poured against a drainage system and waterproofing system fastened to the shoring system.

Waterproofing of the foundation walls is recommended and the membrane is to be installed from 4 m below finished grade down the foundation walls to the bottom of foundation.

It is also recommended that a composite drainage system, such as Delta Drain 6000 or equivalent, be installed between the waterproofing membrane and the foundation wall and extend from the exterior finished grade to the founding elevation (underside of footing or raft slab). The purpose of the composite drainage system is to direct any water infiltration resulting from a breach of the waterproofing membrane to the building sump pit. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the perimeter footing or raft slab interface to allow the infiltration of water to flow to an interior perimeter underfloor drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

It is also recommended that a secondary perimeter foundation drainage system be provided at a depth of 2 m below any frost heave sensitive areas, such as exterior sidewalks for the proposed structure, to control any perched water within the backfill material. The perimeter drainage pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Raft Slab Construction Joints

It is expected that the raft slab, where utilized, will be poured in sections. For the construction joint at each pour, a PVC water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration below the lowest level floor slabs. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centres. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where space is available for conventional wall construction, backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular A, should be used for this purpose.

Pressure Relief Chamber

Where a raft slab is utilized, the pressure relief chamber will be used to control the groundwater infiltration and hydrostatic pressure created by tanking the lower level of underground parking. To avoid uplift on the raft foundation slab prior having sufficient loading to resist uplift, it is recommended that the water infiltration be pumped via the pressure relief chamber during construction.

The valve of the pressure relief chamber can be gradually close during construction as the loading is applied to resist hydrostatic pressure. Once sufficient load is available to resist the full hydrostatic pressure, the valve of the pressure relief chamber can be adjusted and closed to minimize water infiltration volumes. Figure 2 – Pressure Relief Chamber Detail in Appendix 2 provides a schematic of the recommended system.

6.2 Protection of Footing Against Frost Action

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

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

The foundations for the underground parking levels are expected to have sufficient frost protection due to the founding depth. However, it has been our experience that insufficient soil cover is typically provided to entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided for these areas.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. Given the proximity of the underground parking levels to the property lines, it is expected that a temporary shoring will be required to support the excavation for this proposed development.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be 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.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring is anticipated to be required to support the overburden soils for the underground parking levels. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

The temporary shoring system may consist of a soldier pile and lagging system or steel sheet piles which could be cantilevered, anchored or braced. However, in the vicinity of the adjacent building located at 52 Chamberlain Avenue, a stiffer shoring system, such as a secant pile wall, is recommended to support the existing, adjacent building.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 5 – Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m^3	20
Effective 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 are 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.

The temporary shoring system may need tie-back anchors extending onto the adjacent properties. Accordingly, consent agreements should be obtained from the neighboring property owners as well as the City of Ottawa prior to construction.

Excavation Base Stability

The base of supported excavations can fail by three general modes:

☐ Shear failure within the ground caused by inadequate resistance to loads imposed by grade difference inside and outside of the excavation,

☐ Piping from water seepage through granular soils, and

☐ Heave of layered soils due to water pressures confined by intervening low permeability soils.

The potential for base heave in cohesive soils should be determined for stability of flexible retaining systems. The factor of safety with respect to base heave, FS_b, is:

$FS_b = N_b s_u / \sigma_z$

where:

N_b - stability factor dependent upon the geometry of the excavation and given in Figure 1 on the following page.

s_u - undrained shear strength of the soil below the base level

σ_z - total overburden and surcharge pressures at the bottom of the excavation

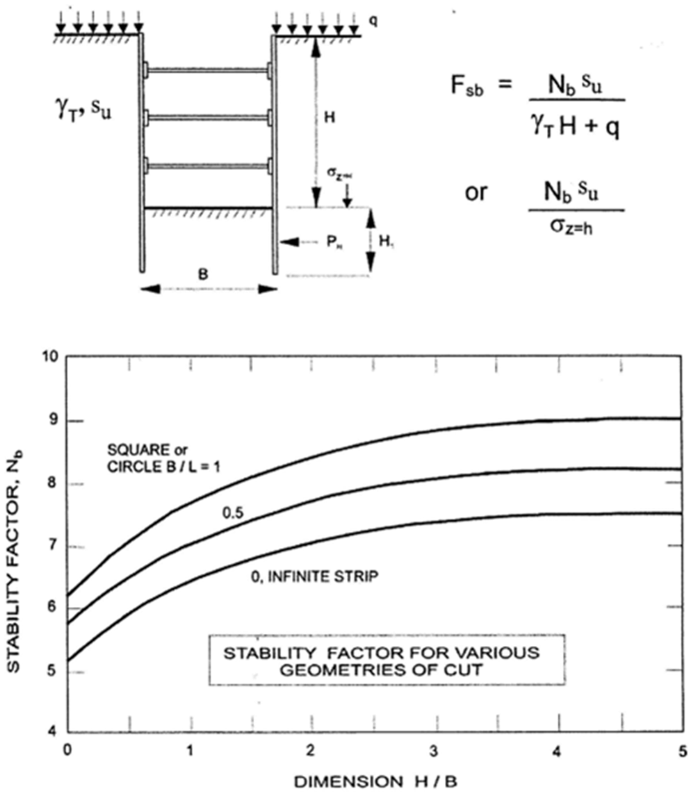


Figure 1 – Stability Factor for Various Geometries of Cut

In the case of stiff clays or compact sands, a factor of safety of 2 is recommended for base stability.

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 sewer and water pipes placed on a relatively dry, undisturbed subgrade surface should consist of at least 150 mm of OPSS Granular A material. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% 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 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

Generally, it should be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay material will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

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) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material’s SPMDD.

To reduce long-term lowering of the groundwater level at this site, clay seals should be installed in the services trenches. The clay seals should be at least 1 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, sub-bedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material’s SPMDD. Due to the small size of the site, clay seals are only recommended to be placed at the site boundaries and at the building interfaces.

6.5 Groundwater Control

Due to the relatively impervious nature of the overlying silty clay within the upper portion of the soil profile, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps.

Where excavations are extended within the sandy silt to silty sand material below the long-term groundwater level, the groundwater infiltration is anticipated to be moderate to high.

Generally, pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. However, if deeper excavations are to occur within the sandy silt to silty sand material below the long-term groundwater level, such as for pits or deep services, consideration should be taken to utilizing a dewatering program consisting of a series of well points designed and installed by a licensed contractor specializing in dewatering.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

A temporary Ministry of 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 of 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, 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. 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.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater which breaches the building's perimeter groundwater infiltration control system will be directed to the sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be very low to negligible. A more accurate estimate of groundwater flow can be provided at the time of construction, once the pressure relief chamber valve is closed and full hydrostatic pressure Is applied to the structure.

Impacts on Neighbouring Properties

Since the proposed building will be founded below the long-term groundwater level, a groundwater infiltration control system has been recommended to lessen the effects of water infiltration. Therefore, long-term dewatering of the site will be minimal and should have no adverse effects to the surrounding buildings or structures. The short-term dewatering during the excavation program will be managed by the excavation contractor, as discussed above.

6.6 Winter Construction

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

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters, 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.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

6.7 Corrosion Potential and Sulphate

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

☐ Review of waterproofing details for elevator shaft(s) and building sump pits.

☐ Review and inspection of the foundation waterproofing and foundation drainage systems.

☐ Review of the pressure relief chamber design and inspection of the installation.

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:

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ Sampling and testing of the concrete and fill materials used.

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation 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 in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request 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 its 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 Quantum Project Management Services Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

O. Canton

Owen R. Canton, B.Eng.

LICENSED PROFESSIONAL ENGINEER
June 3, 2024
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis

Scott S. Dennis, P.Eng.

Report Distribution:

- ☐ Quantum Project Management Services Inc. (1 copy)
- ☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Building - 30-48 Chamberlain Avenue
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5332

REMARKS

HOLE NO. BH 1

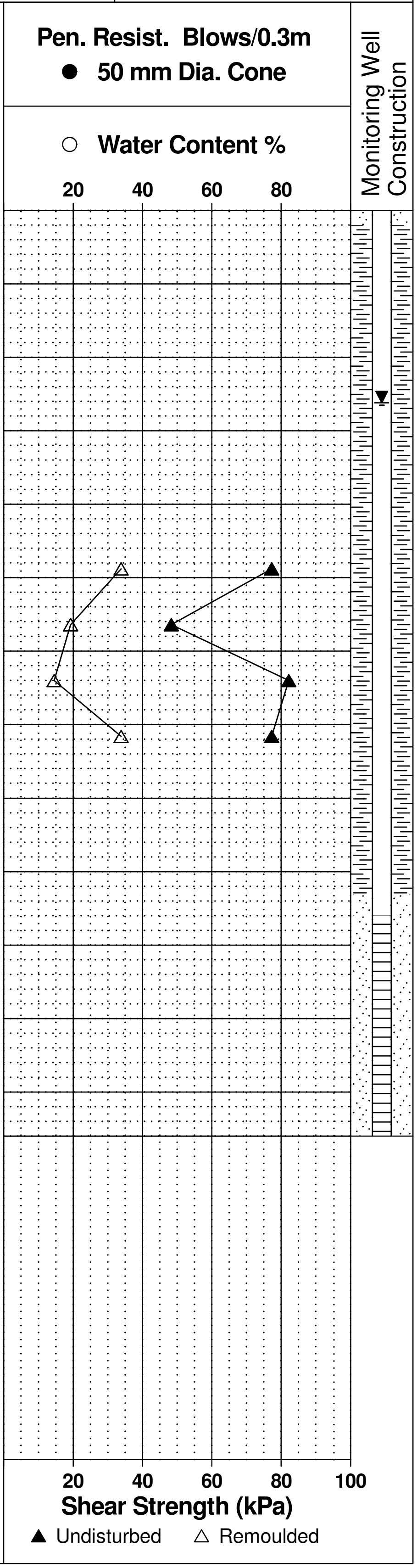
BORINGS BY CME-55 Low Clearance Drill

DATE May 21, 2020

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone	0.53
FILL: Brown silty clay with sand and gravel	1.52
Loose, brown **SILTY SAND**	2.29
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.6m depth	7.62
Grey **SANDY SILT to SILTY SAND,** trace clay	9.30
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	12.60
End of Borehole Practical refusal to augering at 12.60m depth. (GWL @ 2.62m - July 28, 2020)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	8
SS	3	79	9
SS	4	100	6
SS	5	100	5
SS	6	100	4
SS	7	29	6
SS	8	75	2
SS	9	46	4
SS	10		6
SS	11	100	8
SS	12	67	3
SS	13	69	50+

DEPTH (m)	ELEV. (m)
0	67.53
1	66.53
2	65.53
3	64.53
4	63.53
5	62.53
6	61.53
7	60.53
8	59.53
9	58.53
10	57.53
11	56.53
12	55.53



patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

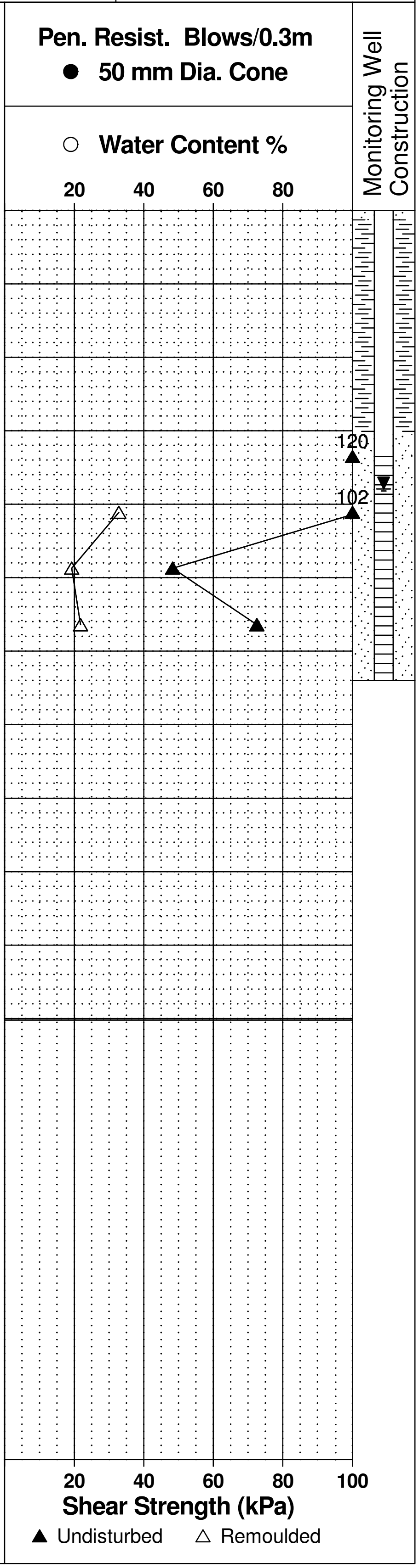
Geotechnical Investigation
Prop. Mixed-Use Building - 30-48 Chamberlain Avenue
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY CME-55 Low Clearance Drill
DATE May 21, 2020
FILE NO. PG5332
HOLE NO. BH 2

SOIL DESCRIPTION	
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone	0.60
FILL: Brown silty sand with gravel, some metal and plastic	1.22
Loose to compact, brown **SILTY SAND**	2.13
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.6m depth	6.25
Grey **SANDY SILT to SILTY SAND,** trace clay	7.92
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	11.02
End of Borehole Practical refusal to augering at 11.02m depth (GWL @ 3.79m - July 28, 2020)	

SAMPLE			
TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	46	7
SS	3	42	13
SS	4	100	5
SS	5	100	W
SS	6	100	14
SS	7	29	7
SS	8	100	4
SS	9	100	3
SS	10	100	50+

DEPTH (m)	ELEV. (m)
0	67.49
1	66.49
2	65.49
3	64.49
4	63.49
5	62.49
6	61.49
7	60.49
8	59.49
9	58.49
10	57.49
11	56.49



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Building - 30-48 Chamberlain Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 21, 2020

FILE NO. PG5332

HOLE NO. BH 3

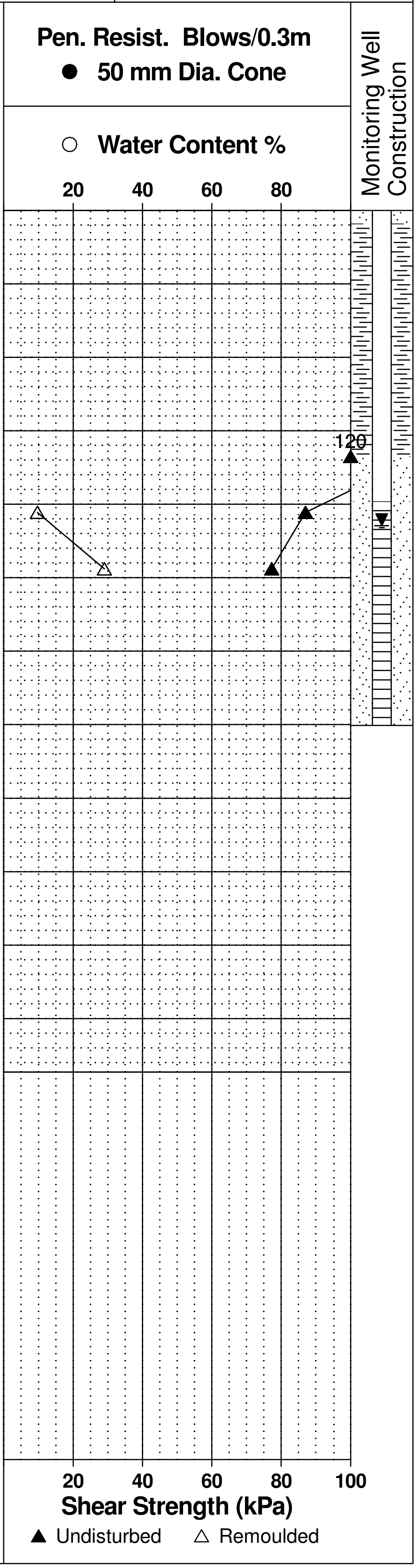
SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10	AU	1		
FILL: Brown silty sand with crushed stone	0.63	SS	2	42	7
Loose, brown **SILTY SAND**	1.52				
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.6m depth - trace sand by 6.4m depth	6.86	SS SS SS	3 4 5	100 100 100	8 5 2
Grey **SANDY SILT to SILTY SAND,** trace clay	9.14	SS SS	6 7	42 100	7 2
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	11.76	SS SS	8 9	38 58	3 8

End of Borehole

Practical refusal to augering at 11.76m depth

(GWL @ 4.29m - July 28, 2020)

DEPTH (m)	ELEV. (m)
0	67.77
1	66.77
2	65.77
3	64.77
4	63.77
5	62.77
6	61.77
7	60.77
8	59.77
9	58.77
10	57.77
11	56.77



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Building - 30-48 Chamberlain Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 22, 2020

FILE NO. PG5332

HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown sand with crushed stone	0.60
FILL: Brown silty sand, some gravel	2.29
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.6m depth	6.55
Grey **SANDY SILT to SILTY SAND,** trace clay	9.14
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	14.73

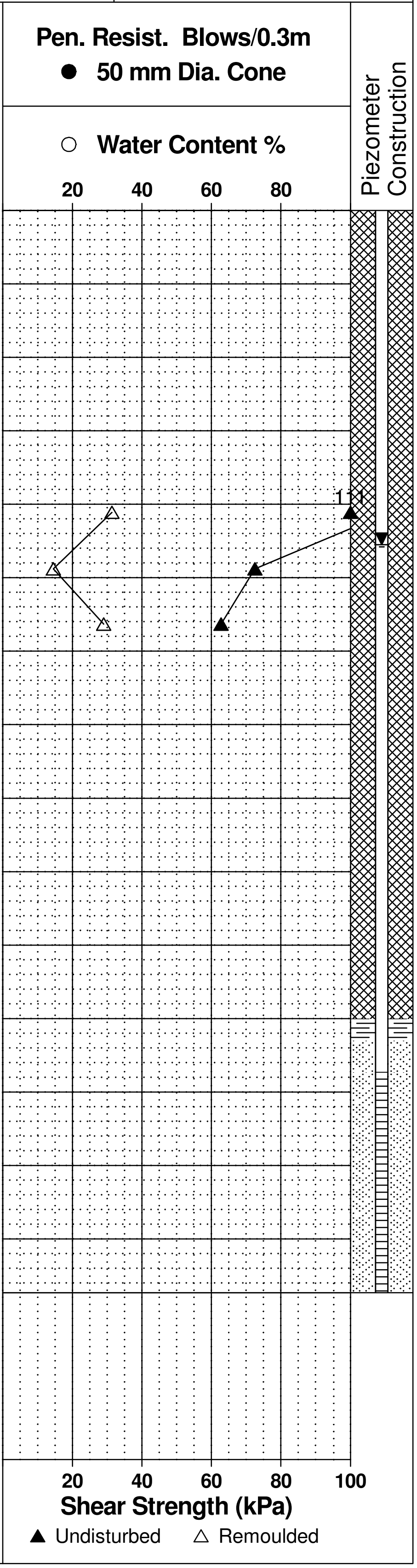
End of Borehole

Practical refusal to augering at 14.73m depth.

(GWL @ 4.55m - May 29, 2020)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	12	5
SS	3	4	8
SS	4	88	7
SS	5	100	3
SS	6	8	P
SS	7	38	6
SS	8	29	5
SS	9	42	3
SS	10	21	9

DEPTH (m)	ELEV. (m)
0	67.48
1	66.48
2	65.48
3	64.48
4	63.48
5	62.48
6	61.48
7	60.48
8	59.48
9	58.48
10	57.48
11	56.48
12	55.48
13	54.48
14	53.48



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Mixed-Use Building - 30-48 Chamberlain Avenue
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE May 22, 2020

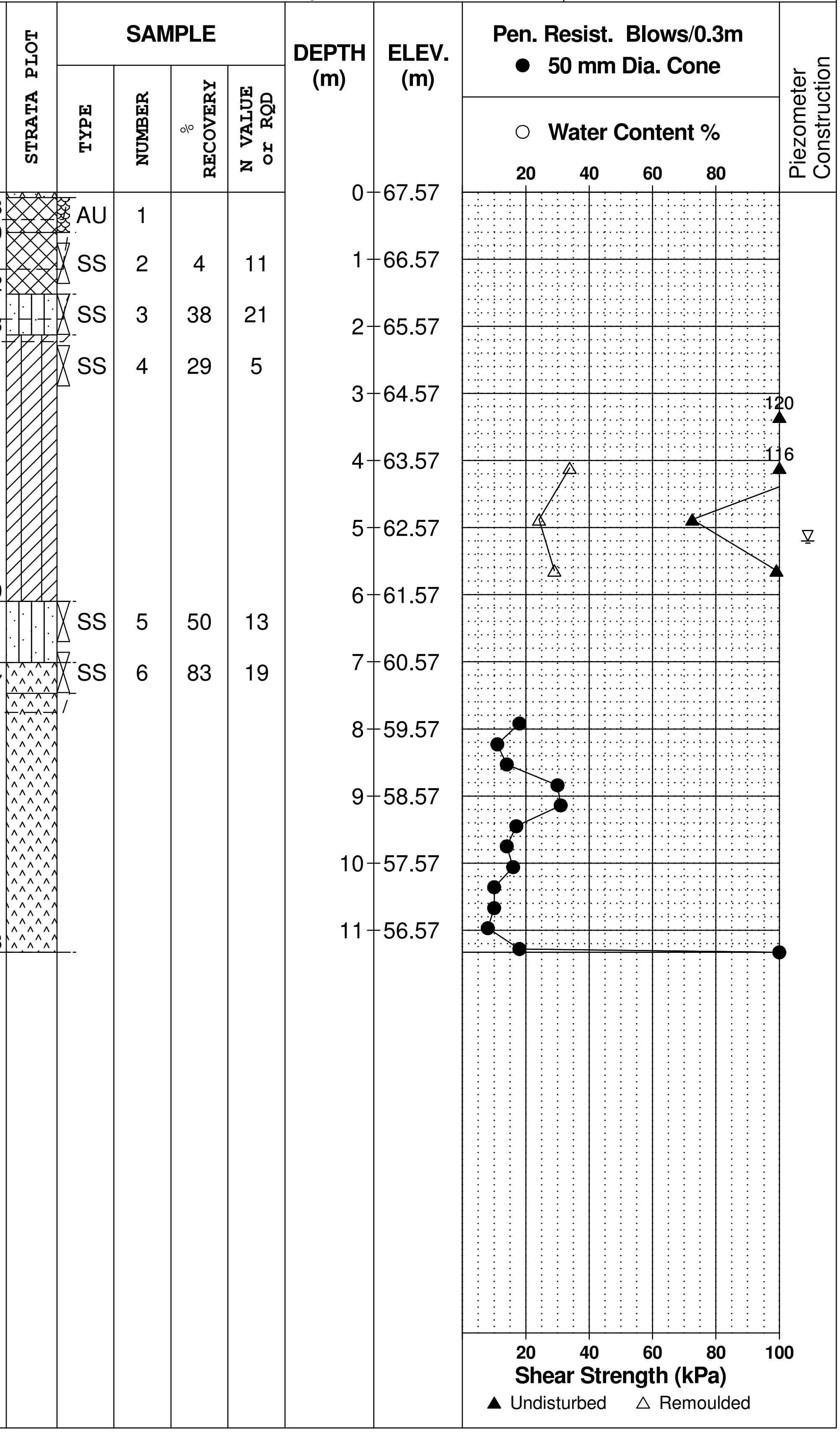
FILE NO. PG5332

HOLE NO. BH 5

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone	0.60
FILL: Brown silty clay, some sand, gravel and brick	1.52
Compact, brown **SILTY SAND**	2.13
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.6m depth	6.10
Grey **SANDY SILT to SILTY SAND,** trace clay	7.01
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	7.47
Dynamic Cone Penetration Test commenced at 7.47m depth. Inferred **GLACIAL TILL**	11.33
End of Borehole Practical DCPT refusal at 11.33m depth (GWL @ 5.2m depth based on field observations)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	4	11
SS	3	38	21
SS	4	29	5
SS	5	50	13
SS	6	83	19

DEPTH (m)	ELEV. (m)
0	67.57
1	66.57
2	65.57
3	64.57
4	63.57
5	62.57
6	61.57
7	60.57
8	59.57
9	58.57
10	57.57
11	56.57



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic Limit, % (water content above which soil behaves plastically)
PI - Plasticity Index, % (difference between LL and PL)

Dxx - Grain size at which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 x 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)

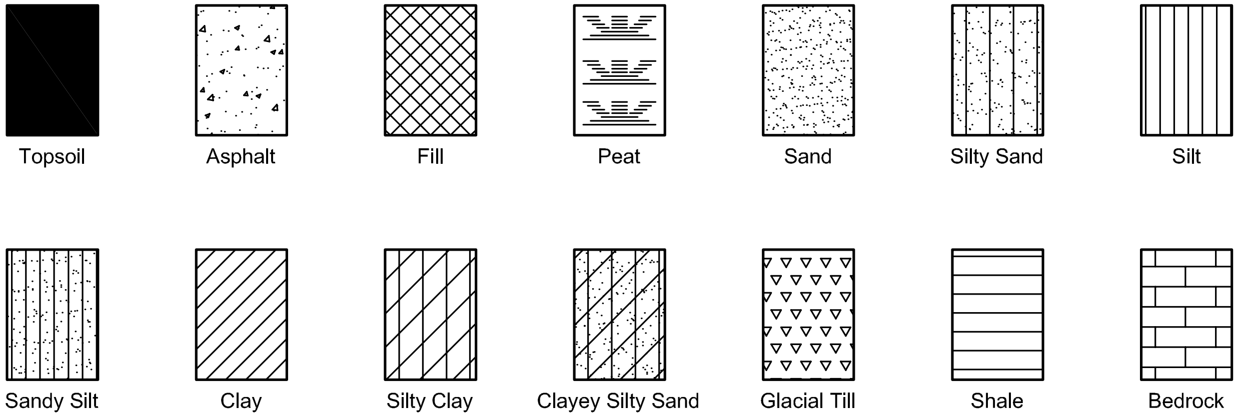
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

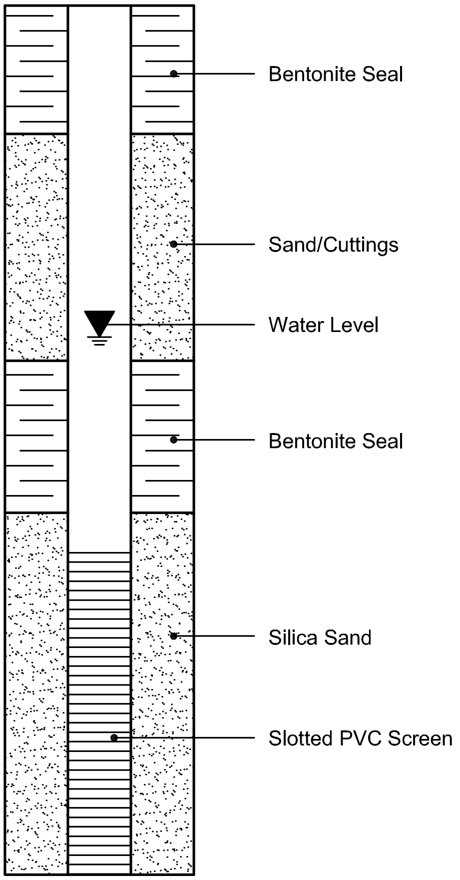
SYMBOLS AND TERMS (continued)

STRATA PLOT

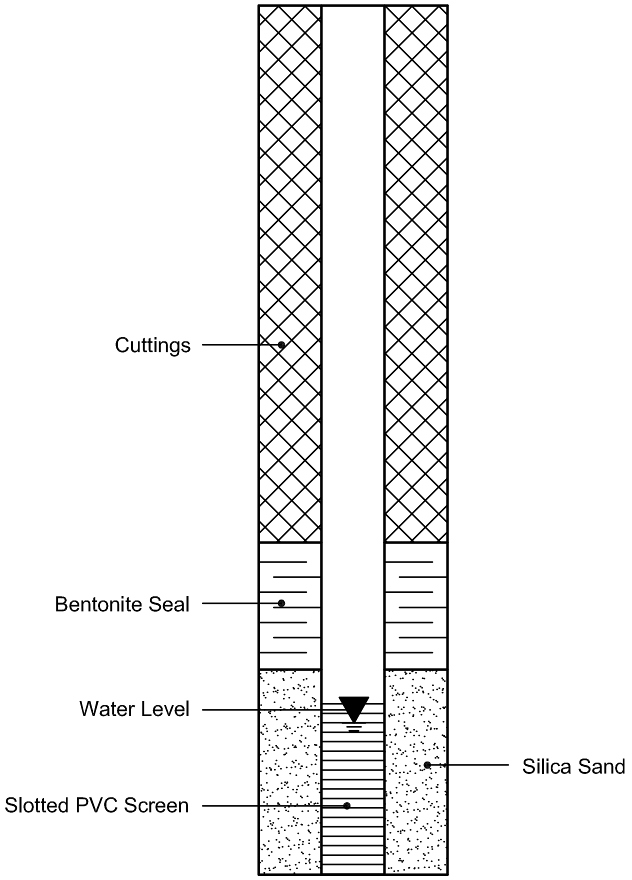


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 01-Jun-2020

Client: Paterson Group Consulting Engineers

Order Date: 26-May-2020

Client PO: 30153

Project Description: PG5332

	Client ID:	BH1-SS6-12'6-14'6	-	-	-
	Sample Date:	21-May-20 11:00	-	-	-
	Sample ID:	2022150-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	63.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.11	-	-	-
Resistivity	0.10 Ohm.m	40.3	-	-	-
Anions					
Chloride	5 ug/g dry	65	-	-	-
Sulphate	5 ug/g dry	88	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 – PRESSURE RELIEF CHAMBER DETAIL

DRAWING PG5332-1 – TEST HOLE LOCATION PLAN

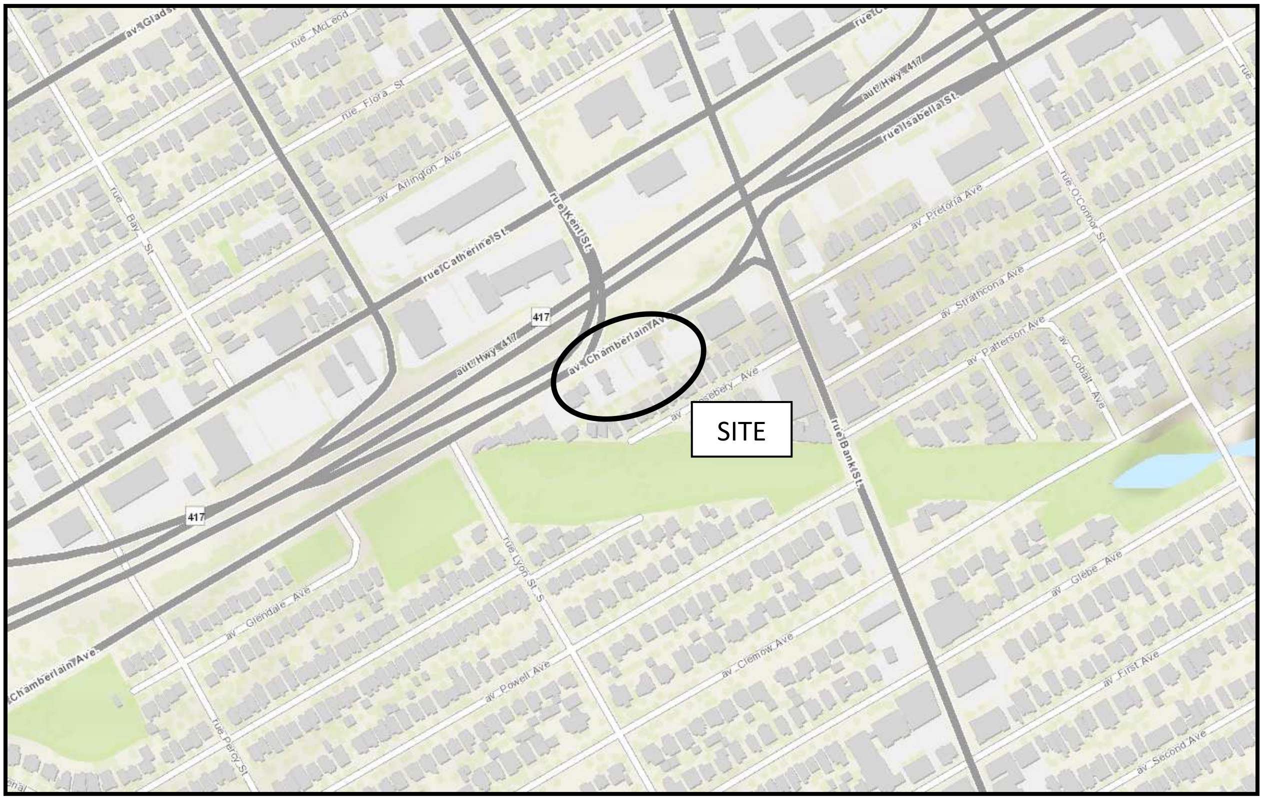


FIGURE 1

KEY PLAN

PATERSON
GROUP

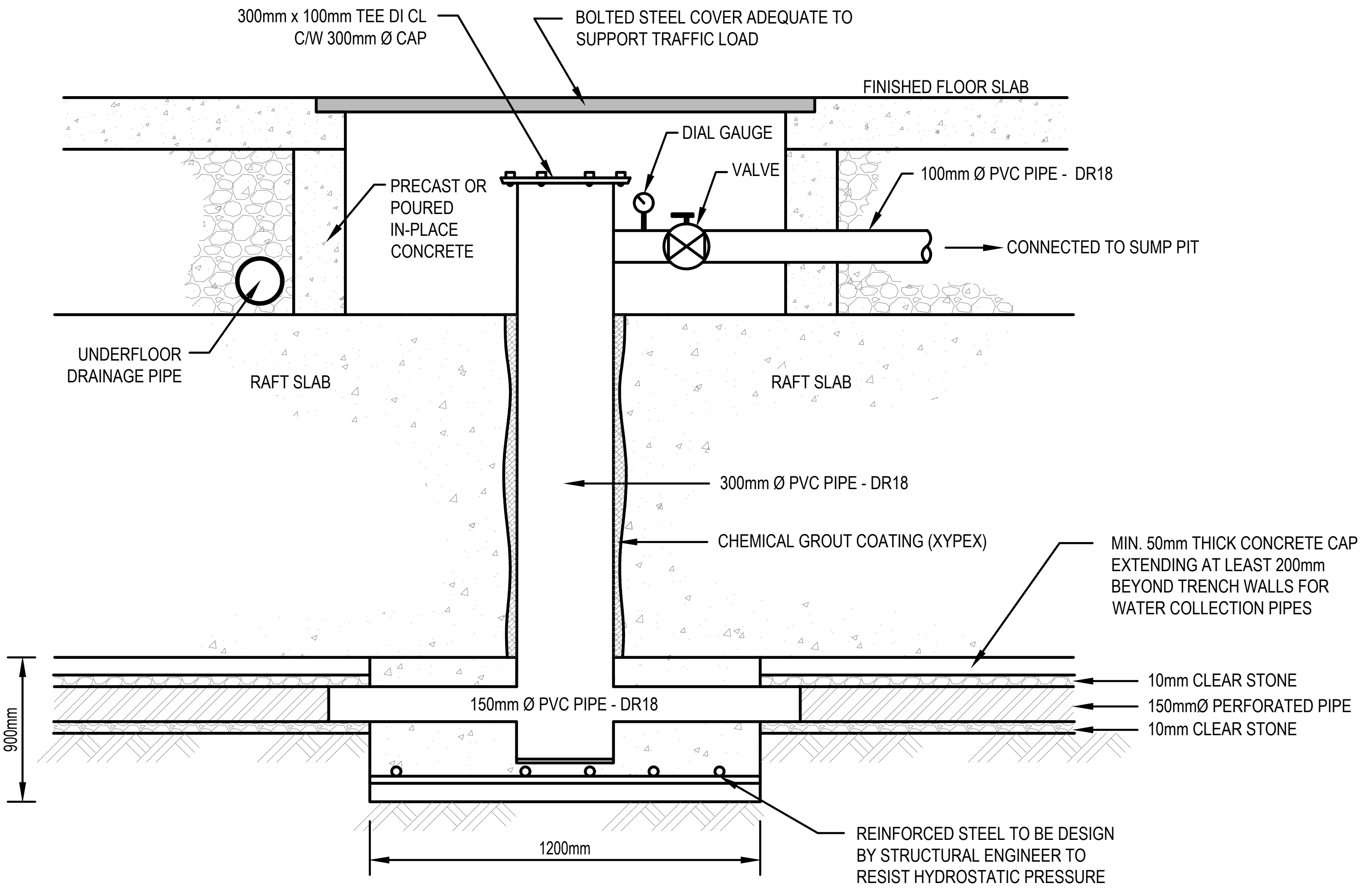


FIGURE 1 - PRESSURE RELIEF CHAMBER

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

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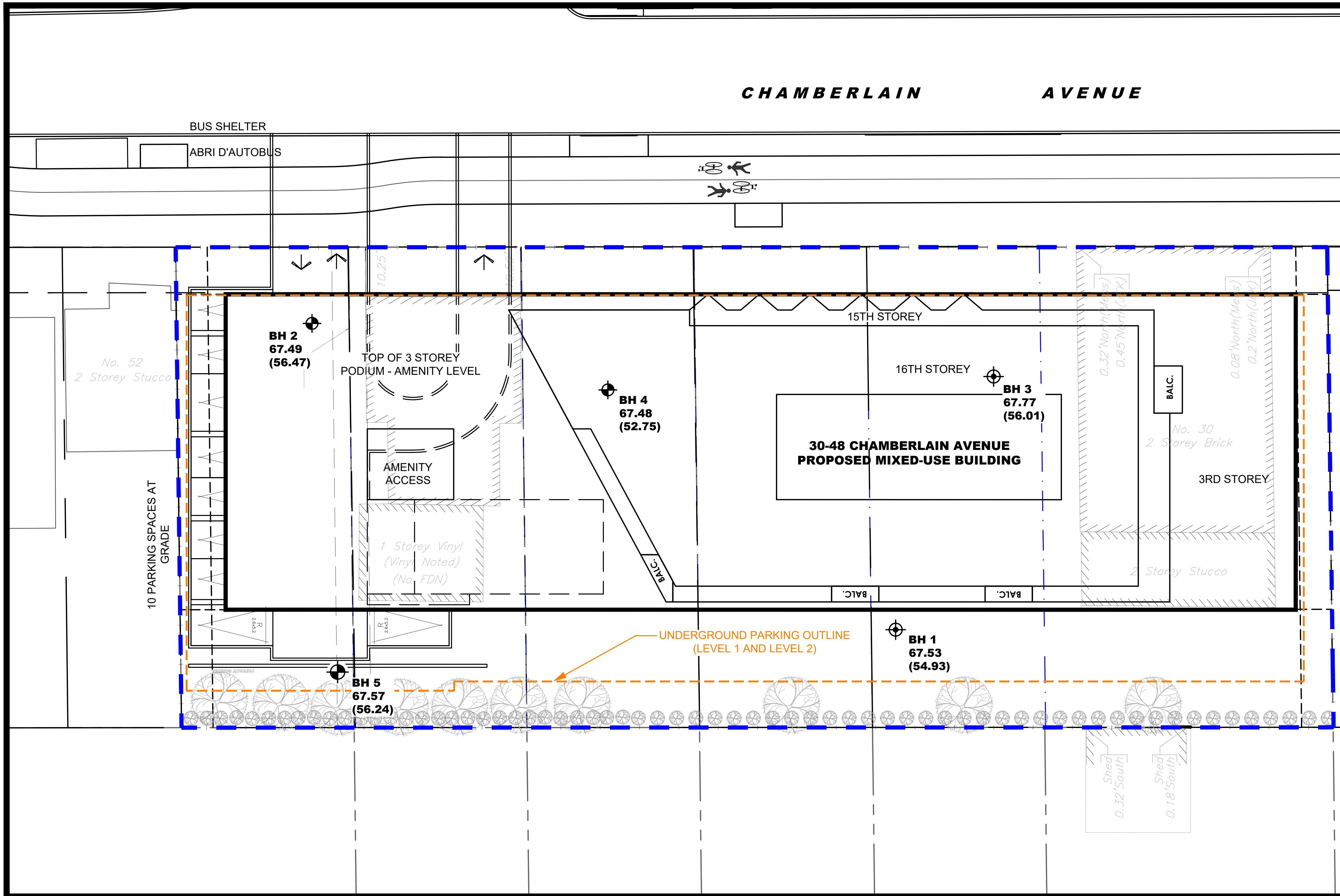

LEGEND:

⌖	BOREHOLE LOCATION
⌖	BOREHOLE WITH MONITORING WELL LOCATION
67.53	GROUND SURFACE ELEVATION (m)
(54.93)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

BASE PLAN PROVIDED BY FARLEY, SMITH AND DENIS SURVEYING LTD.

CONCEPTUAL PLAN PROVIDED BY HOBIN ARCHITECTURE

BOREHOLE ELEVATIONS ARE REFERENCED TO GEODETIC DATUM.

SCALE: 1:250

0 1 2 3 4 5 10 15m

PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO NEW CONCEPTUAL PLAN	11/01/2023	YZ

QUANTUM PROJECT MANAGEMENT SERVICES INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE BUILDING
30 - 48 CHAMBERLAIN AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250
Date: 01/2023
Drawn by: YA
Report No.: PG5332-1
Checked by: DP
Dwg. No.: PG5332-1
Approved by: SD
Revision No.: 1

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