Geotechnical Engineering

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Geotechnical Investigation

Proposed Multi-Storey Building
15 Oblats Avenue
Ottawa, Ontario

Prepared For

Domicile Developments

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Report PG5329-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Domicile Developments to conduct a geotechnical investigation for the proposed multi-storey building to be located at 15 Oblats Avenue, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2 of this report).

The objectives of the investigation were to:

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

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

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. A report addressing environmental issues for the subject site was prepared under separate cover.

2.0 Proposed Development

Based on the available information, it is understood that the subject site is located at the northern portion of the Springhurst Park, at the the intersection of Oblats Avenue and Deschatelets Avenue.

Details of the proposed developments were not available at the time of writing this report. However, It is our understanding that the proposed project will consist of a multi-storey building with one (1) underground level occupying the open portions of the subject site. Associated at-grade parking areas, access lanes and landscaped areas are further anticipated. It is expected that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on April 22, 2020 and consisted of four (4) boreholes which were advanced to a maximum depth of 6.1 m below existing ground surface. A previous investigation was completed by this firm within the boundaries of the subject site consisting of one borehole (BH 3) located along the west property line of the subject site. The borehole was advanced to a maximum depth of 9.6 m below existing grade. The borehole locations were determined in the field by Paterson personnel taking into consideration site features and underground services. The locations of the boreholes are shown on Drawing PG5329-1 - 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 test hole procedures consisted of advancing the boreholes to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using three different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All soil samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags and 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.

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

Undrained shear strength testing was carried out in cohesive soils at BH 3-20 and BH 4-20 using a field vane apparatus.

Overburden thickness was evaluated during the course of the site investigation by dynamic cone penetration testing (DCPT) at the location of BH 4-20. 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, BH 4 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. Similarly, a flexible standpipe was installed in BH 3 to permit monitoring of the groundwater table within this location.

Sample Storage

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

3.2 Field Survey

The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson. The ground surface elevations at the borehole locations were surveyed with respect to a temporary benchmark (TBM), consisting of the top of spindle of the fire hydrant located on the northeast side of the property. A geodetic elevation of 64.54 m was provided for the TBM by Farley, Smith and Denis Surveying. The borehole locations and ground surface elevation at each borehole location are presented on Drawing PG5329-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the boreholes and visually examined in our laboratory to review the field logs.

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 Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a three (3) storey institutional building with one (1) basement level and the remainder of the site is occupied by parking lots, grass covered areas and mature trees. The site is bordered to the north by Springhurst Avenue, to the south by Oblats Avenue, to the east by Parish Private. The site is bordered by single family homes to the north and east, and multi-storey buildings to the west. The site is relatively flat with a slight downslope towards the west and approximately at grade with neighbouring properties and adjacent roadways.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the borehole locations consist of a thin layer of topsoil and fill material, consisting of brown silty sand with trace cobbles, crushed stone, and/or organics, overlying a silty clay deposit. The upper portion of the silty clay deposit consists of a stiff, weathered brown silty clay crust overlying a stiff, grey silty clay. Practical refusal to DCPT was encountered at BH 4 at a depth of 25.58 m. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the bedrock in this area consists of dark brown to black shale of the Billings Formation and gray shale of the Carlsbad Formation with an overburden drift thickness of 15 to 50 m depth.

4.3 Groundwater

Groundwater levels were measured at the monitoring wells in the borehole locations of the current investigation on April 24, 2020. The measured groundwater levels in the piezometers at the borehole locations are presented in Table 1. The long term groundwater level can also be estimated based on the recovered soil samples' moisture levels and consistency. Based on these observations, the long term groundwater table is anticipated to be at a 3 to 4 m depth. It should be further noted that the groundwater level could vary at the time of construction.

Table 1 - Summary of Groundwater Levels

Borehole Number	Measured Groundwater Level		Recording Date
	Depth (m)	Elevation (m)	
Groundwater Levels Based on Current Investigation (Report PG5329)			
BH 1	3.54	62.29	April 24, 2020
BH 2	2.67	61.96	April 24, 2020
BH 4	3.83	60.46	April 24, 2020
Groundwater Levels Based on Previous Investigation (Report PG2973)			
BH 3	3.23	60.59	May 31, 2013

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is satisfactory for the proposed development. The proposed building can be founded on conventional style shallow foundations placed on an undisturbed, stiff silty clay or engineered fill bearing surface. If the proposed multi-storey building is expected to hold more than 3 storeys, options such as raft foundation placed on an undisturbed stiff silty clay bearing surface or end bearing piled foundations should be considered due to the anticipated loading being too excessive for conventional shallow footings.

Due to the presence of a silty clay deposit, the subject site will be subjected to a permissible grade raise restriction.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, asphalt, and deleterious fill, such as material containing high content of organic materials, should be stripped from under the proposed buildings footprint and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the proposed buildings 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.

5.3 Foundation Design

Bearing Resistance Values (3 storey building or less)

Strip footings, up to 3 m wide, and pad footings, up to 6 m wide, placed in an undisturbed, stiff silty clay bearing or engineered backfill surface can be designed using a bearing resistance value at SLS of **125 kPa** and a factored bearing resistance value at ULS of **225 kPa**.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings.

The bearing resistance value at SLS given for footings 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 with respect to excavations and different foundation levels. Adequate lateral support is provided to a stiff silty clay 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.

Raft Foundation (4 to 9 storey Building)

Alternatively, consideration can be given to a raft foundation if the building loads exceed the bearing resistance values provided for a conventional shallow footing foundation. The following parameters may be used for raft design.

It is understood that the base of the raft foundation will be located between 5 to 6.5 m below grade. Due to the raft shape, the underside of the raft will be placed between geodetic elevations of 57.8 to 60.8 m.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The bearing resistance value at SLS (contact pressure) of **180 kPa** will be considered acceptable. 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. The contact pressure provided considers the stress relief associated with the soil removal required for proposed building. The factored bearing resistance (contact pressure) at ULS can be taken as **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **5 MPa/m** for a contact pressure of **180 kPa**. The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

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.

End Bearing Piled Foundation (Greater than 9 Storeys)

A deep foundation method, such as end bearing piles, can also be considered for the proposed structure if the design building loads exceed the bearing resistance values provided for a conventional shallow footing or raft foundation. Concrete filled steel pipe piles driven to refusal on a bedrock surface are a typical deep foundation option in Ottawa.

Applicable pile resistance at SLS values and factored pile resistance at ULS values are provided in Table 1. Additional resistance values can be provided if available pile sizes vary from those detailed in Table 1. 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 calculating the Hiley dynamic formula. The piles should be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of four piles is recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - End Bearing Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)		Final Set (blows/ 25 mm)	Transferred Hammer Energy (kJ)
		Factored at ULS (kN)		
245	10	1460	10	35.9
245	12	1650	10	42
245	13	1760	10	45.4

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 pile driving 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

Consideration must be given to potential settlements which could occur due to the presence of the silty clay deposit and the combined loads from the proposed footings, any groundwater lowering effects, and grade raise fill. The foundation loads to be considered for the settlement case are the continuously applied loads which consist of the unfactored dead loads and the portion of the unfactored live load that is considered to be continuously applied. A minimum value of 50% of the live load is often recommended by Paterson.

A permissible grade raise restriction of **1 m** is recommended for finished grading within 5 m of the proposed building. A post-development groundwater lowering of 1 m was considered in our permissible grade raise restriction calculations.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for the shallow foundations at the subject site. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and deleterious material, containing organic matter, within the footprint of the proposed building, the approved existing fill or native soil surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

It is expected that the basement area will be mostly parking and a rigid pavement structure designed by a structural engineer will be applicable. However, if storage or other uses of the lower level where a lean concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab 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 the minimum 98% of its SPMDD.

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. 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 (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 0.32g 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

Where required at the subject site, the recommended pavement structures for car only parking areas and access lanes are shown 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

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 II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, such as Terratrack 200 or equivalent, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable 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.

Due to the impervious nature of the subgrade materials consideration should be given to installing subdrains during the pavement construction. These drains should be installed at each catch basin, be at least 3 m long and should extend in four orthogonal directions or longitudinally when placed along a curb. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 150 mm diameter perforated 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. An interior perimeter drainage pipe should be placed along the building perimeter along with a sub-floor drainage system. The perimeter drainage pipe and under-floor drainage system should direct water to sump pit(s) within the lower garage area.

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 I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) 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 heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

Unsupported Excavations

The side slopes of excavations in the soil and fill overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soils at this site 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.

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

If a temporary shoring system is considered, 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 potential for a fully saturated condition following a significant precipitation event. Any changes to the approved shoring design system should be reported immediately to the owner’s representative prior to implementation.

Temporary shoring may be required to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures, and the elevation of the adjacent building foundations and underground services. Additional information can be provided when the above details are known.

For design purposes, the temporary system may 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 added to the earth pressures described below. These systems can be cantilevered, anchored or braced.

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

Table 5 - Soil Parameters for Shoring System Design

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
Total Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ), kN/m^3	13

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure.

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

The design of the rock anchors for temporary shoring can be based on the values provided in Table 6. From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes.

Table 6 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	4	1.2	5.2	250
	5.6	1.7	7.3	500
	7.9	2.4	10.3	1000
125	3.9	1.1	5	250
	5.3	1.6	6.9	500
	7.2	2.2	9.4	1000

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

Soldier Pile and Lagging System

The active earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of 0.65 K γ H for strutted or anchored shoring or a triangular earth pressure distribution with a maximum value of K γ H for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component.

Underpinning

Founding conditions of adjacent structures bordering the site should be assessed and underpinning requirements should be evaluated.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & 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 should consist of a minimum of 150 mm of OPSS Granular A material. Where the bedding is located within the firm grey silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. The material should be placed in a maximum 300 mm thick loose lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extent 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 a maximum 300 mm thick loose lifts and compacted to a minimum of 95% of its SPMDD.

It should generally 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 materials 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 a 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 provided in the service trenches. The seals should be at least 1.5 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, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

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

It is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of the shallow excavation. 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.

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. 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 encountered along the building's perimeter or sub-slab drainage system will be directed to the proposed building's sump pit. It is expected that groundwater flow will be low (i.e.- less than 25,000 L/day) with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

It is understood that one (1) underground level maybe included for the proposed building. Based on the existing groundwater level and low permeability of the native soils, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures. It should be noted that no issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

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

6.7 Corrosion Potential and Sulphate

The results on analytical testing show that the sulphate content is less than 0.1%. The results are indicative that Type 10 Portland 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 in indicative of a moderate to slightly aggressive corrosive environment.

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that the following material testing and observation program be performed by the geotechnical consultant.

☐ A review of the final grading plan should be completed from a geotechnical perspective.

☐ 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 material testing and observation program by the geotechnical consultant.

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 of this nature 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 Domicile Developments or their agents is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Faisal I. Abou-Seido, P.Eng.

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
May 27, 2020
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Report Distribution

- ☐ Domicile Developments (e-mail 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
Proposed Multi-Storey Buidling - 15 Oblats Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the northeast corner of subject site. Geodetic elevation = 64.54m.

REMARKS

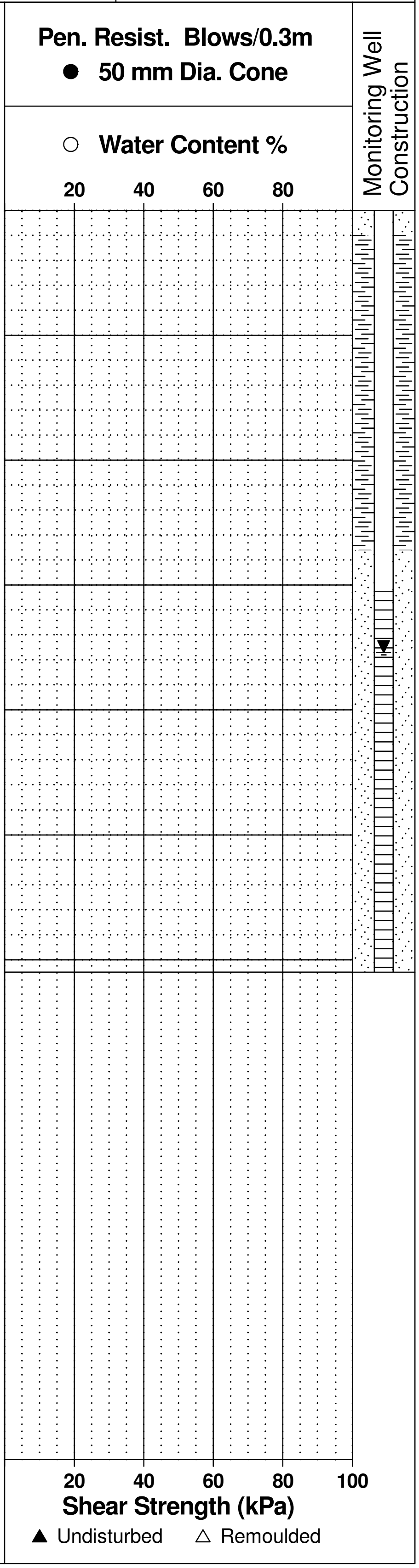
BORINGS BY Track Mount Power Auger

DATE April 22, 2020

FILE NO. PG5329

HOLE NO. BH 1-20

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	65.83
TOPSOIL 0.08						
FILL: Brown silt, some clay, trace sand and organics	AU	1				
					1	64.83
	SS	2	4	9		
1.45						
Compact, brown **SILTY SAND**	SS	3	42	10		
					2	63.83
2.29						
Stiff, grey-brown **SILTY CLAY**	SS	4	100	3		
					3	62.83
- grey by 3.0m depth	SS	5	100	W		
					4	61.83
	SS	6	100	2		
					5	60.83
	SS	7	100	W		
	SS	8	100	W		
6.10					6	59.83
End of Borehole						
(GWL @ 3.54m - April 24, 2020)						



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buidling - 15 Oblats Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the northeast corner of subject site. Geodetic elevation = 64.54m.

REMARKS

BORINGS BY Track Mount Power Auger

DATE April 22, 2020

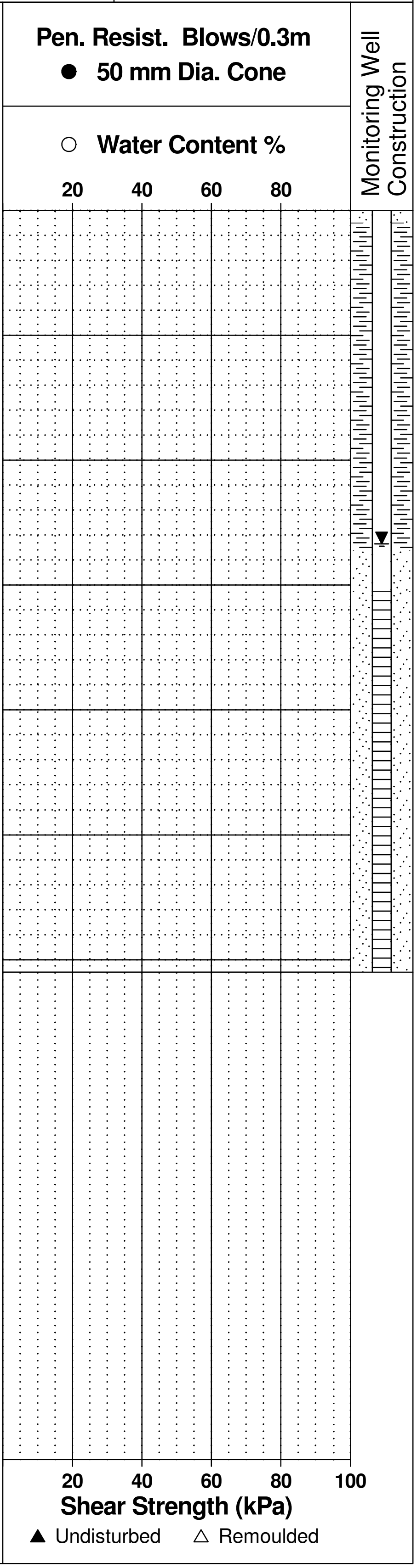
FILE NO. PG5329

HOLE NO. BH 2-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.63
TOPSOIL 0.08							
FILL: Compact, brown silty sand		AU	1				
1.20		SS	2	25	9	1	63.63
Loose, brown **SILTY SAND**		SS	3	54	5		
2.00						2	62.63
Stiff, brown **SILTY CLAY,** trace sand		SS	4	33	2		
		SS	5	100	2	3	61.63
		SS	6	100	2	4	60.63
		SS	7	100	2	5	59.63
		SS	8	100	2		
6.10						6	58.63

End of Borehole

(GWL @ 2.67m - April 24, 2020)



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buidling - 15 Oblats Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the northeast corner of subject site. Geodetic elevation = 64.54m.

REMARKS

BORINGS BY Track Mount Power Auger

DATE April 22, 2020

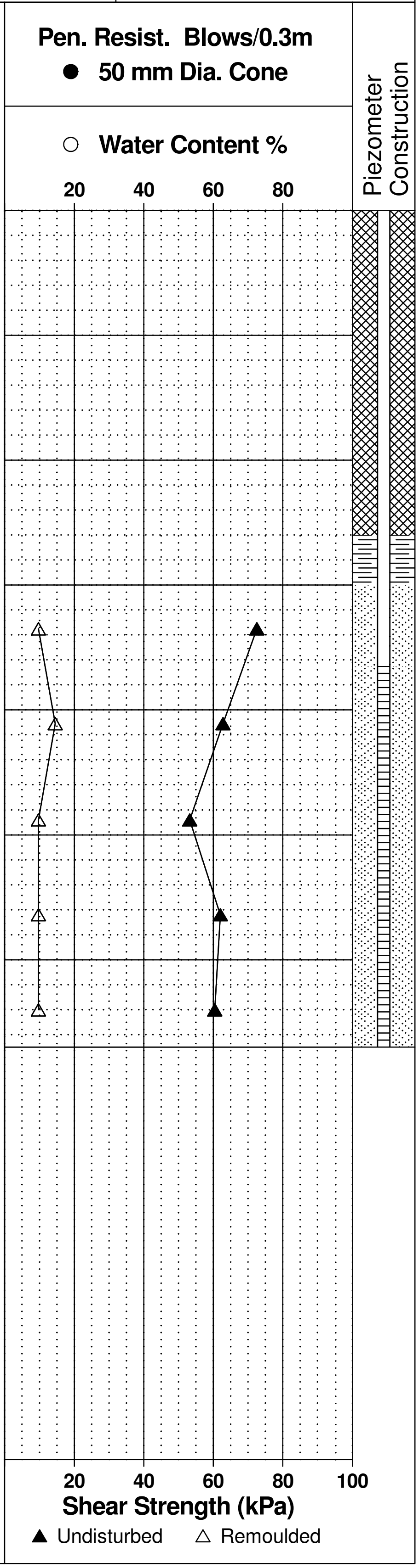
FILE NO. PG5329

HOLE NO. BH 3-20

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
TOPSOIL	0.05
FILL: Dark brown silt, sand, some organics	1.10
Loose, brown **SILTY SAND**	2.06
Stiff, brown **SILTY CLAY**	6.70
End of Borehole	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	33	9
SS	3	29	8
SS	4	79	3

DEPTH (m)	ELEV. (m)
0	65.34
1	64.34
2	63.34
3	62.34
4	61.34
5	60.34
6	59.34



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buidling - 15 Oblats Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the northeast corner of subject site. Geodetic elevation = 64.54m.

REMARKS

BORINGS BY Track Mount Power Auger

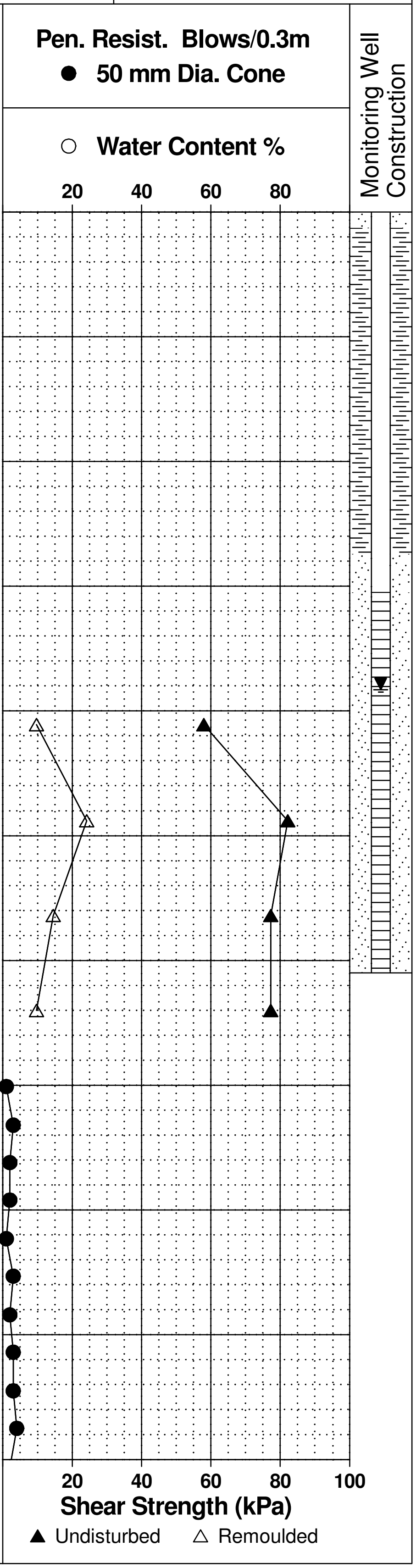
DATE April 22, 2020

FILE NO. PG5329

HOLE NO. BH 4-20

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Asphaltic concrete 0.10	AU	1		
FILL: Grey-brown silt, sand and organics, trace asphaltic concrete	SS	2	17	9
	SS	3	21	5
	SS	4	29	2
3.05				
Stff, grey **SILTY CLAY**	SS	5	100	1
6.70				
Dynamic Cone Penetration Test commenced at 6.70m depth.				

DEPTH (m)	ELEV. (m)
0	64.29
1	63.29
2	62.29
3	61.29
4	60.29
5	59.29
6	58.29
7	57.29
8	56.29
9	55.29
10	54.29



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buidling - 15 Oblats Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the northeast corner of subject site. Geodetic elevation = 64.54m.

REMARKS

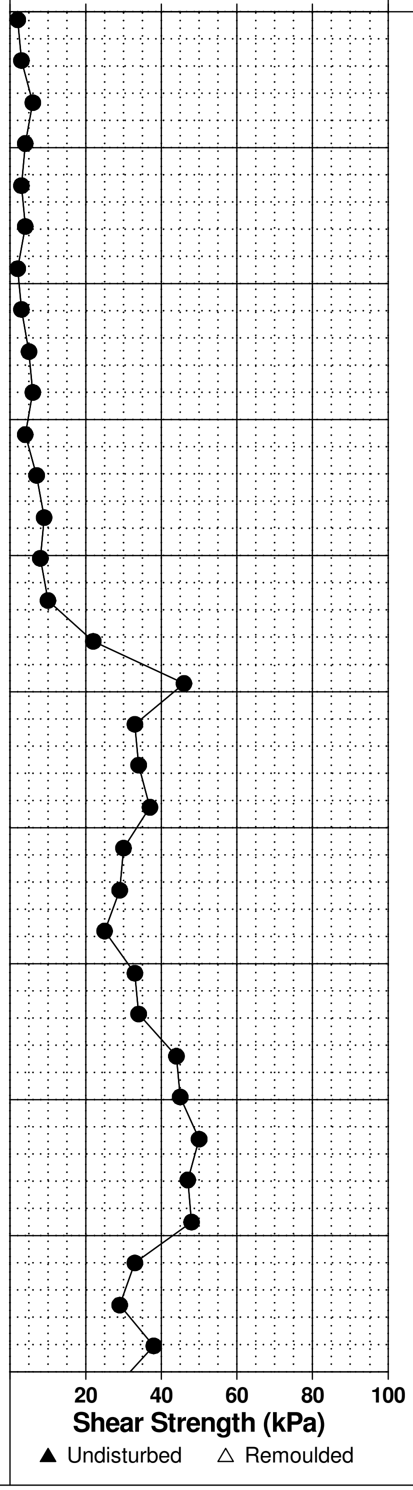
BORINGS BY Track Mount Power Auger

DATE April 22, 2020

FILE NO. PG5329

HOLE NO. BH 4-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE									
						10	54.29		
						11	53.29		
						12	52.29		
						13	51.29		
						14	50.29		
						15	49.29		
						16	48.29		
						17	47.29		
						18	46.29		
						19	45.29		
						20	44.29		



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buidling - 15 Oblats Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the northeast corner of subject site. Geodetic elevation = 64.54m.

REMARKS

BORINGS BY Track Mount Power Auger

DATE April 22, 2020

FILE NO. PG5329

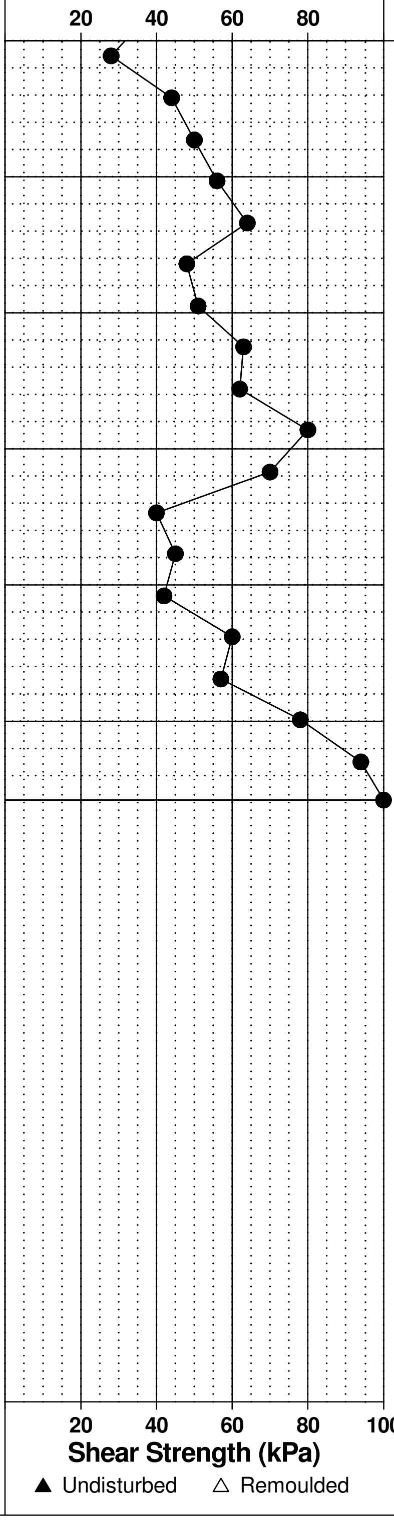
HOLE NO. BH 4-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						20	44.29
						21	43.29
						22	42.29
						23	41.29
						24	40.29
						25	39.29
25.58							

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

○ Water Content %

Monitoring Well Construction



End of Borehole

Practical DCPT refusal at 25.58m depth.

(GWL @ 3.83m - April 24, 2020)

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buildings - 15 Oblate Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, located on the northeast corner of the subject site on the south side of Springhurst Avenue, in front of subject site. Geodetic elevation = 64.54m, as per Farley, Smith and Denis Surveying Ltd.

REMARKS

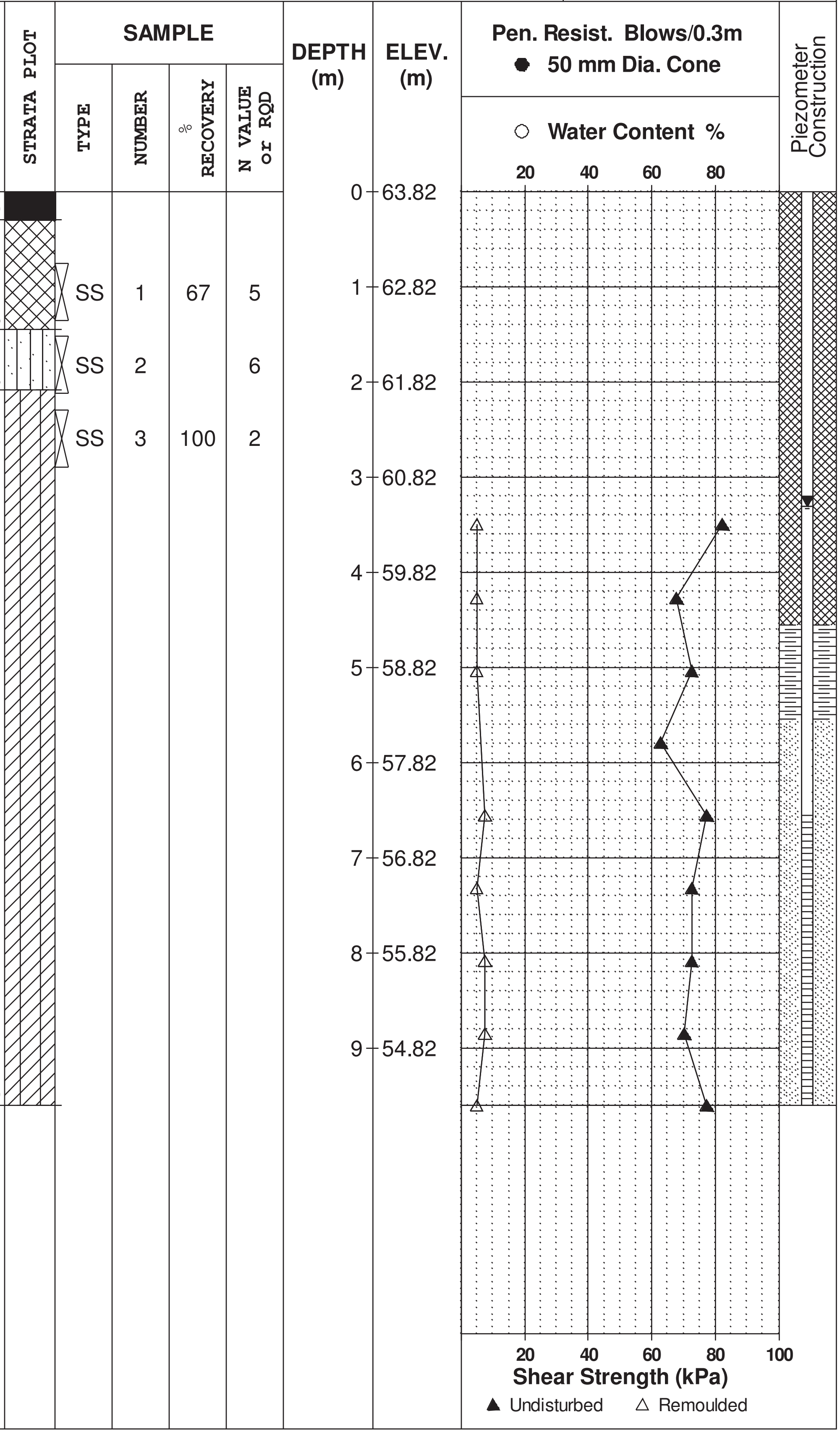
BORINGS BY CME 55 Power Auger

DATE May 16, 2013

FILE NO. PG2973

HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	63.82
TOPSOIL 0.30						
FILL: Brown clayey silt with sand 1.45	SS	1	67	5	1	62.82
Loose, brown **SILTY SAND** with clay seams 2.08	SS	2		6	2	61.82
Stiff, brown **SILTY CLAY** - grey by 5.9m depth 9.60	SS	3	100	2	3	60.82
					4	59.82
					5	58.82
					6	57.82
					7	56.82
					8	55.82
					9	54.82
End of Borehole (GWL @ 3.31m-May 31, 2013)						



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = $D60 / D10$

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

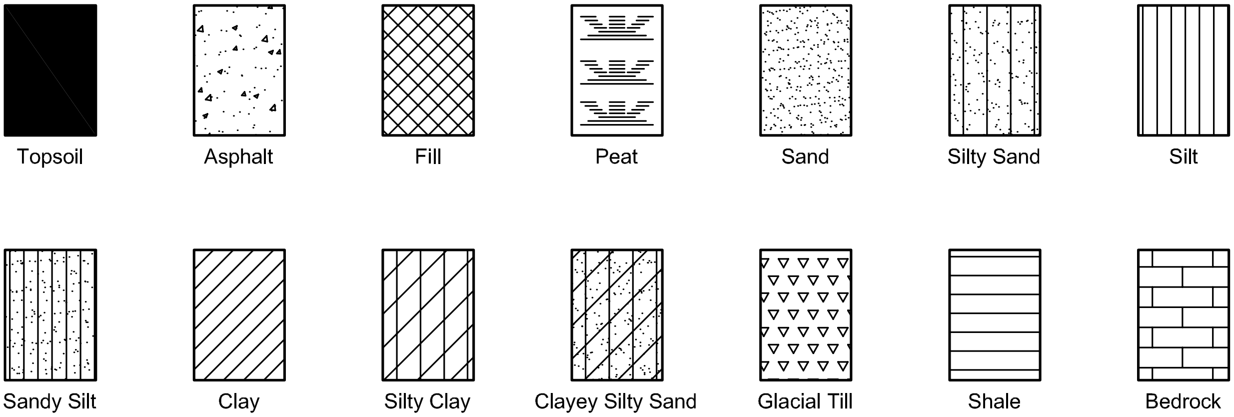
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

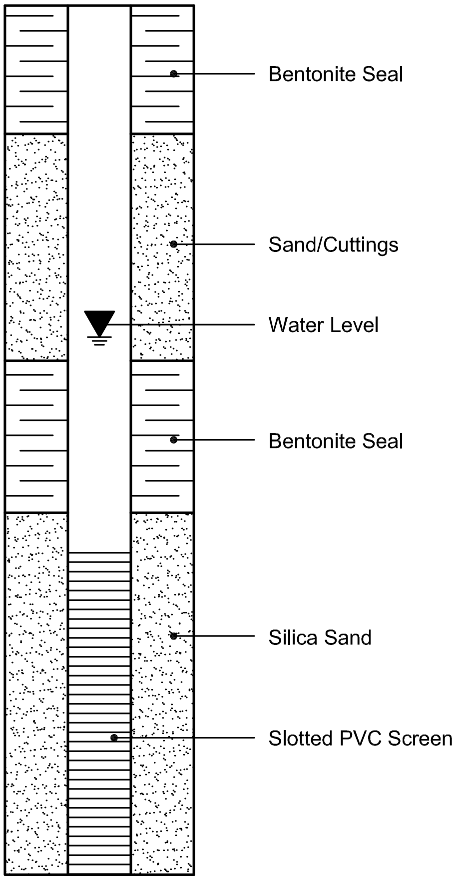
SYMBOLS AND TERMS (continued)

STRATA PLOT

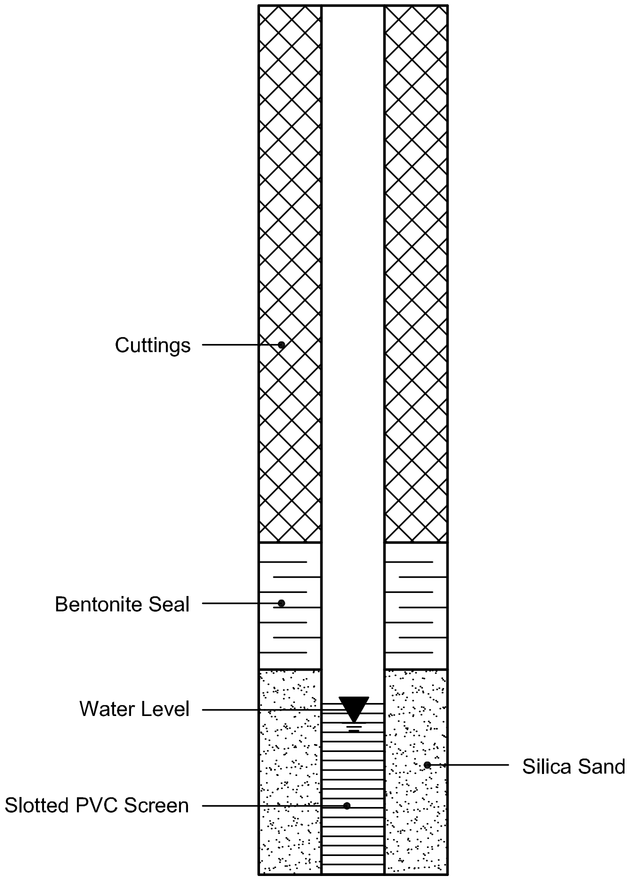


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 24-May-2013
Order Date:17-May-2013

Client: **Paterson Group Consulting Engineers**
Client PO: 13969
Project Description: PG2973

	Client ID:	BH3-SS3	-	-	-
	Sample Date:	16-May-13	-	-	-
	Sample ID:	1320265-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	66.8	-	-	-
General Inorganics					
pH	0.05 pH Units	7.29	-	-	-
Resistivity	0.10 Ohm.m	21.6	-	-	-
Anions					
Chloride	5 ug/g dry	179	-	-	-
Sulphate	5 ug/g dry	429	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5329-1 - TEST HOLE LOCATION PLAN

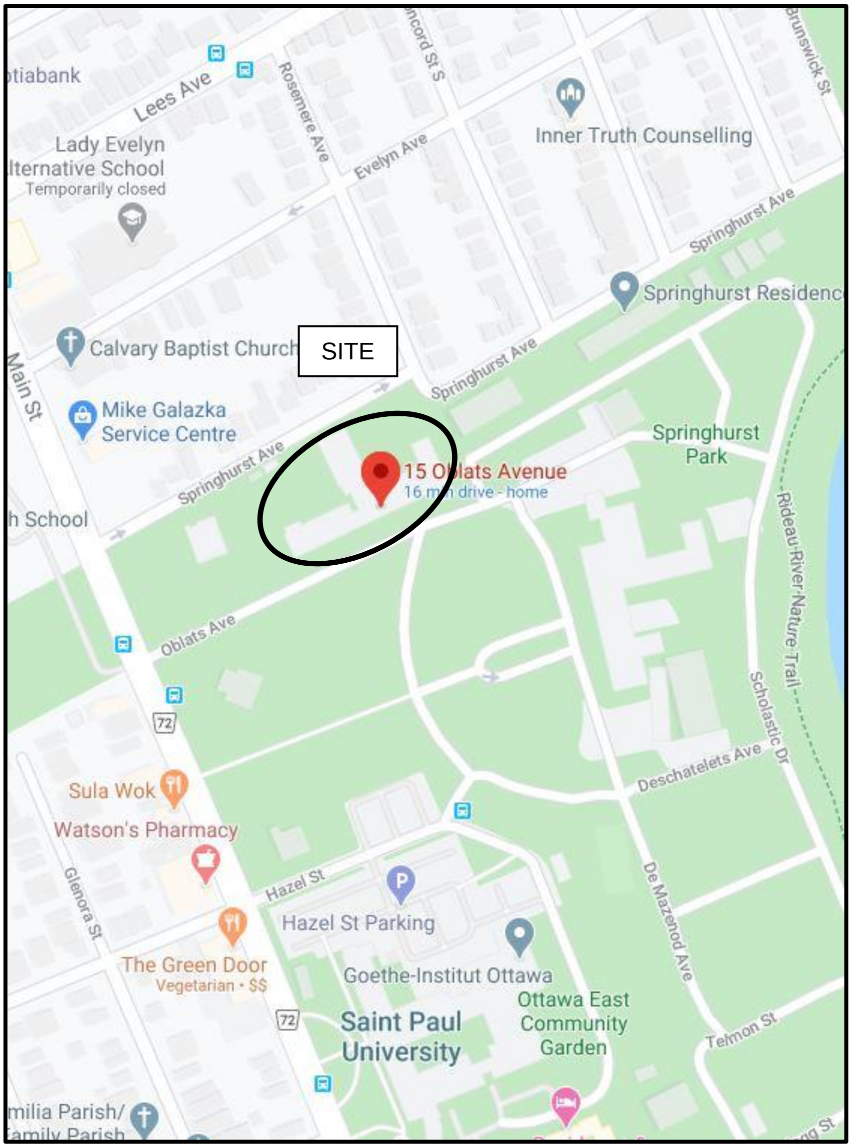
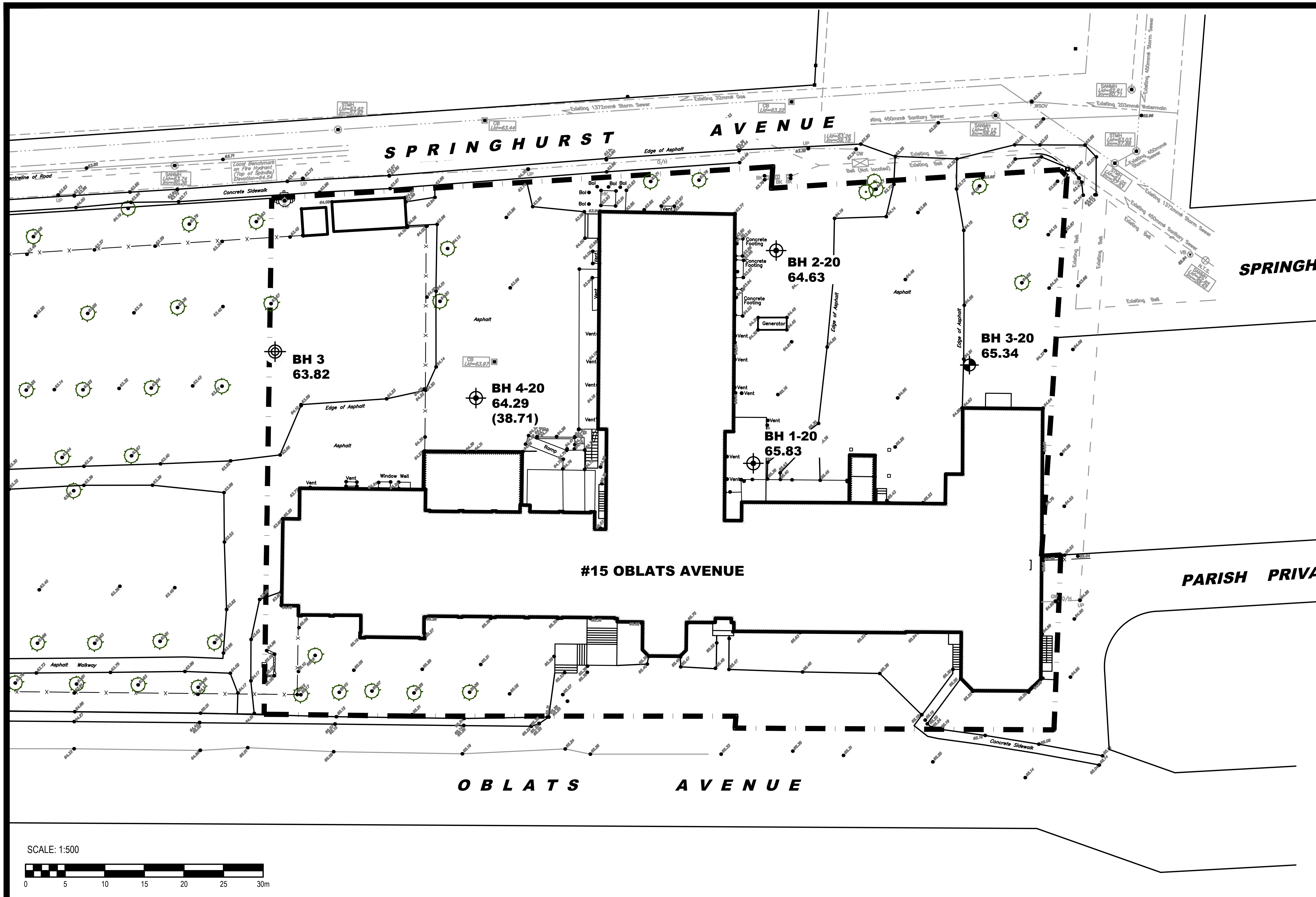


FIGURE 1

KEY PLAN



LEGEND:

⌖ BOREHOLE WITH MONITORING WELL LOCATION

⌖ BOREHOLE LOCATION

⌖ BOREHOLE LOCATION, PATERSON GROUP REPORT PG2973, 2013

64.29 GROUND SURFACE ELEVATION (m)

(38.71) PRACTICAL DCPT REFUSAL ELEV. (m)

TBM - TOP SPINDLE OF FIRE HYDRANT LOCATED NEAR THE NORTHEAST CORNER OF SUBJECT SITE. GEODETIC ELEVATION = 64.54

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
0			

DOMICILE DEVELOPMENTS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING - 15 OBLATS AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500
Date: 05/2020
Drawn by: MPG
Report No.: PG5329-1
Checked by: YT
Dwg. No.: PG5329-1
Approved by: DJG
Revision No.:

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