Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

patersongroup

Geotechnical Investigation

Proposed Residential Building
347 Gilmour Street and
278, 280 & 282 O'Connor Street
Ottawa, Ontario

Prepared For

AK Global Management Inc.

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

August 1, 2019

Report: PG4799-1 Revision 1

Table of Contents

Appendices

Appendix 1	Soil Profile and Test Data Sheets Symbols and Terms Analytical Testing Results
Appendix 2	Figure 1 - Key Plan Drawing PG4799-1 - Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by AK Global Management Inc. to conduct a geotechnical investigation for the proposed multi-storey residential building to be located at 347 Gilmour Street and 278, 280 and 282 O'Connor Street, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the investigation was to:

- ❏ determine the subsoil and groundwater conditions at this site by means of test holes and available soils information.
- ❏ provide geotechnical recommendations for the design of the proposed development based on the results of the test holes and other soil information available.

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

2.0 Proposed Project

Based on available plans, the proposed project will consist of a 6 storey building, with one underground parking level, as well as associated access roads and landscaped areas. It is also expected that the subject site 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 January 24 and 25, 2019. At that time, three (3) boreholes (BH 1 to BH 3) were advanced to a maximum depth of 9.6 m below existing ground surface. The test holes were located in the field by Paterson in a manner to provide general coverage of the subject site. The borehole locations are shown on Drawing PG4799-1 - Test Hole Location Plan in Appendix 2.

The boreholes were drilled with a truck-mounted rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department. The drilling procedures consisted of advancing each test hole to the required depths at the selected locations and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples from the borehole were recovered from the auger flights or using a 50 mm diameter split-spoon sampler. All soil samples were classified on site, placed in sealed plastic bags and transported to our laboratory for further review. The depths at which the auger and split spoon samples were recovered from the test hole are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

Standard Penetration Testing (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, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The overburden thickness was also evaluated during the course of the investigation by dynamic cone penetration testing (DCPT) at two borehole locations. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its 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

Flexible polyethylene standpipes were installed within all three boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

All samples from the current investigation 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 were selected in the field by Paterson personnel in a manner to provide general coverage of the subject site taking into consideration existing site features. Ground surface elevations were referenced to a temporary benchmark (TBM), consisting of the top of a manhole cover located near the site on Gilmour Street. A geodetic elevation of 71.16 m was provided for the TBM based on the topographic survey plan prepared by Farley, Smith & Denis Surveying Ltd. The location of the test holes and TBM are presented on Drawing PG4799-1 - 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 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 an asphalt surfaced parking area and associated access lanes for the existing dwellings occupying the site. The site is bordered to the north and west by existing residential buildings, to the south by Gilmour Street and to the east by O'Connor Street. The site has a gentle gradient down from north to south across the existing parking area.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the borehole locations consists of an asphalt pavement structure, overlying a brown, very stiff to stiff silty clay deposit. Firm, grey silty clay was encountered below the stiff clay layer. Practical refusal to DCPT was encountered at 18.8 and 19.0 m depth in BH 1 and BH 3, respectively. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the local bedrock consists of shale of the Billings formation with an anticipated overburden thickness of between 15 to 25 m.

4.3 Groundwater

Groundwater levels were measured in the standpipes installed in the boreholes upon completion of the sampling program. The groundwater level readings are presented in Table 1 and on the Soil Profile and Test Data sheets in Appendix 1.

Table 1 - Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level (m)		Recording Date
		Depth	Elevation	
BH 1	71.95	2.23	69.72	January 31, 2019
BH 2	71.94	3.04	68.90	January 31, 2019
BH 3	71.74	Blocked	-	January 31, 2019

It should be noted that groundwater levels can be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater table can be expected at approximately 4 to 5 m below ground surface.

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

The subject site is considered satisfactory for the proposed development, from a geotechnical perspective. Based on the available information at the time of preparation of this report, it is expected that the building loads will exceed soil bearing resistance values, if conventional footings are considered. Bearing resistance values for conventional footings are provided herein, however it is recommended that a raft slab foundation be used for the proposed structure, placed on an undisturbed, stiff silty clay bearing surface, or a pile foundation driven to a bedrock bearing surface.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, containing deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the subgrade level during site preparation activities.

Due to the depth of the proposed underground parking levels, it is understood that the fill layer encountered will be completely removed from within the proposed building footprint.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, 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 should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

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

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 6 m wide, placed on an undisturbed, stiff brown silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa** incorporating a geotechnical resistance factor of 0.5 at ULS.

The bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Raft Foundation

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

For design purposes, it was assumed that the base of the raft foundation for the proposed multi-storey building will be located at a 3 to 4 m depth with one anticipated underground level.

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 **150 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 the proposed building. The factored bearing resistance (contact pressure) at ULS can be taken as **225 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 **2.0 MPa** for a contact pressure of **150 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

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 stiff silty clay or engineered fill above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passing through in situ soil of the same or higher bearing capacity as the bearing medium soil.

Permissible Grade Raise

Considering the close proximity of the proposed building to the property line, placement of additional fill surrounding the building is not anticipated. However, a permissible grade raise restriction of **2 m** can be used for design purposes.

Pile Foundation

A deep foundation system driven to refusal in the bedrock could be considered for support of the proposed building. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance values at ULS are given in Table 2. 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 vlaues were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

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)
		Factored at ULS (kN)		
245	9	1495	25	40
245	11	1750	24	48.5
245	13	2000	25	56

The minimum centre-to-centre pile spacing is 2.5 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.

Lateral Load Resistance

Lateral loads on the foundations can be resisted using passive resistance on the sides of the foundations. For Limit States Design, the resistance factor to be applied to the ultimate lateral resistance, including passive pressure, is 0.50. The total lateral resistance will be comprised of the individual contributions from up to several material layers, as follows.

Geotechnical parameters for the in-situ fill and for typical backfill materials compacted to 98% of SPMDD in 300 mm lift thicknesses are provided in Table 3, below, along with the associated earth pressure coefficients for horizontal resistance calculations for footings under lateral loads or deadman anchors. Friction factors between concrete and the various subgrade materials are also provided in Table 3, where normal loads allow them to be used.

Where granular soils and/or granular backfill materials are present, the passive pressure can be calculated using a triangular distribution equal to $K_P \cdot \gamma \cdot H$ where:

K_P = factored passive earth pressure coefficient of the applicable retained soil, 1.5
γ = unit weight of the fill of the applicable retained soil (kN/m^3)
H = height of the equivalent wall or footing side (m)

Note that for cases where the depth to the top of the structure (i.e. footing) pushing against the soil does not exceed 50% of the depth to the base of the structure, the effective value of H in the above noted relationship will be the overall depth to the base of the structure. There will also be "edge effects" where the effective width of soil providing the resistance can be increased by 50% of the effective depth on each side of the pushing structural component.

Note that where the foundation extends below the groundwater level, the effective unit weight should be utilized for the saturated portion of the soil.

Should additional passive resistance be required, the horizontal component of the axial resistance of battered piles (up to 1H:3V inclination), or anchors can be used in the building foundation design.

Foundation Uplift Resistance

Uplift forces on the proposed foundations can be resisted using the dead weight of the concrete foundations, the weight of the materials overlying the foundations, and the submerged weight of the piles. Unit weights of materials are provided in Table 3.

For soil above the groundwater level, calculate using the "drained" unit weight and below groundwater level use the "effective" unit weight. Backfilled excavations in low permeability soils can be expected to fill with water and the use of the effective unit weights would be prudent if drainage of the anchor footings is not provided.

As noted, the piles will generally be located below the groundwater level, so the submerged, or effective, weight of the pile will be available to contribute to the uplift resistance, if required. A resistance factor of 0.9 is applicable for the ULS weight component.

Table 3 - Geotechnical Parameters for Uplift and Lateral Resistance Design

Material Description	Unit Weight (kN/m³)		Internal Friction Angle (°) φ'	Friction Factor, tan δ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_A	At-Rest K_O	Passive K_P
OPSS Granular A Fill (Crushed Stone)	22.0	13.7	38	0.60	0.22	0.36	8.8
OPSS Granular B Type I Fill (Well-Graded Sand-Gravel)	21.5	13.4	36	0.55	0.26	0.41	7.5
OPSS Granular B Type II Fill (Crushed Stone)	22.5	14.0	40	0.62	0.20	0.33	10.3
Granular Working Surface - Coarse Open-Graded Crushed Stone	19.0	12.0	36	0.55	0.26	0.41	7.5
Site Excavated Silty Sand or Gravelly Sand Fill	18.0	11.2	32	0.48	0.30	0.46	5.6

Notes:
- Properties for fill materials are for condition of 98% of standard Proctor maximum dry density.
- The earth pressure coefficients provided are for horizontal backfill profile.
- Passive pressure coefficients incorporate wall friction of 0.5 φ'.

A sieve analysis and standard Proctor test should be completed on each of the fill materials proposed to obtain an accurate soil density to be expected, so the applicable unit weights can be estimated.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for the foundations considered at this site. The soils underlying the proposed shallow foundations 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 fill, containing organic matter, within the footprint of the proposed building, the native soil surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II is recommended for backfilling below the basement slab. All backfill materials within the footprint of the proposed building should be placed in maximum 300 mm loose lifts and compact to at least 98% of the material's SPMDD.

It is expected that the basement area for the proposed multi-storey building will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level are proposed where a concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone.

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, in our opinion, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

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

Static Earth Pressures

The static horizontal earth pressure (P_o) can be calculated by 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)

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) can be calculated using the earth pressure distribution equal to $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ 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. The vertical seismic coefficient is assumed to be zero.

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

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

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

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

5.7 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of car only parking areas and access lanes.

Table 4 - 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 silty clay or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 5 - Recommended Pavement Structure - Access Lanes 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 silty clay 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.

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 extend in four orthogonal directions or longitudinally when placed along a curb. The clear crushed stone surrounding the drainage lines or the pipe, should be wrapped with suitable filter cloth. 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

Perimeter Drainage System

Based on the preliminary information provided, it is expected that a portion of the proposed building foundation walls will be below the long-term groundwater table. To limit long-term groundwater lowering, it is recommended that a groundwater infiltration control system be designed for the proposed building. Also, a perimeter foundation drainage system will be required as a secondary system to account for any groundwater which breaches the primary groundwater infiltration control system.

The groundwater infiltration control system should extend at least 1 m above the long-term groundwater level. The following is suggested for preliminary design purposes:

- ❏ Place a suitable membrane against the temporary shoring surface, such as a bentomat liner system or equivalent. The membrane liner should extend down to footing level. The membrane liner should also extend horizontally a minimum of 600 mm below the footing at underside of footing level.
- ❏ Place a composite drainage layer, such as Delta Drain 6000 or equivalent, over the membrane as a secondary system. The composite drainage layer should extend from finished grade to underside of footing level.
- ❏ Pour the foundation wall against the composite drainage system.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration below the underground parking structure. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

It is recommended that 150 mm diameter sleeves, spaced at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of any water that breaches the waterproofing system to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Foundation Backfill

Where space is available, 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 composite drainage system, such as Delta Drain 6000 or an approved equivalent. 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 effects of frost action. A minimum 1.5 m thick soil cover (or equivalent) should be provided in this regard.

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for exterior unheated footings, not thermally connected to a heated space, such as exterior columns and/or wing walls.

It has been our experience that insufficient soil cover is typically provided to footings located in areas where minimal soil cover is available, such as entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided.

6.3 Excavation

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

Unsupported Excavation

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 soil is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. 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.

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

It is recommended that a trench box be used at all times to protect personnel working in trenches 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 will be required for the overburden soil 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.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer 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 system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, the shoring systems should be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is the preferred method.

The earth pressures acting on the shoring system may be calculated with the following parameters.

Table 6 - 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.

A 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, a minimum factor of safety of 1.5 should be used.

Due to the depth of the proposed building excavation and proximity to adjacent properties, underpinning of the existing building foundations may be required to ensure the lateral support zone is maintaned. Underpinning requirements for adjacent nearby buildings should be assessed at the time of construction.

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 at least 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 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 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 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 maximum 225 mm thick loose layers 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

It is anticipated that groundwater infiltration into the excavations should be low 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 which encounters the building's perimeter groundwater infiltration control system will be directed to the proposed building's sump pit. It is expected that groundwater flow will be low (i.e. less than 10,000 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 two levels of underground parking are planned for the proposed building. Based on the existing groundwater level and low permeability of the adjacent 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

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

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

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost into the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions.

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

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

- ❏ A review of the site grading plan(s) from a geotechnical perspective, once available.
- ❏ A review of architectural and structural drawings to ensure adequate frost protection is provided to the subsoil.
- ❏ 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 the construction has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

The recommendations provided in this report are intended for the use of design professionals associated with this project. Contractors bidding on or undertaking the work should examine the factual information contained in this report and the site conditions, satisfy themselves as to the adequacy of the information provided for construction purposes, supplement the factual information if required, and develop their own interpretation of the factual information based on both their and their subcontractors construction methods, equipment capabilities and schedules.

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 AK Global Management Inc. or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Aug. 1-2019
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Nathan F. S. Christie, P.Eng. Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ❏ AK Global Management Inc. (3 copies)
- ❏ 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
347 Gilmour Street, 278, 280 & 282 O'Connor Street
Ottawa, Ontario

DATUM TBM - Top of grate of manhole cover located on Gilmour Street, near the southeast corner of subject site. Geodetic elevation = 71.16m.

REMARKS

BORINGS BY Geoprobe

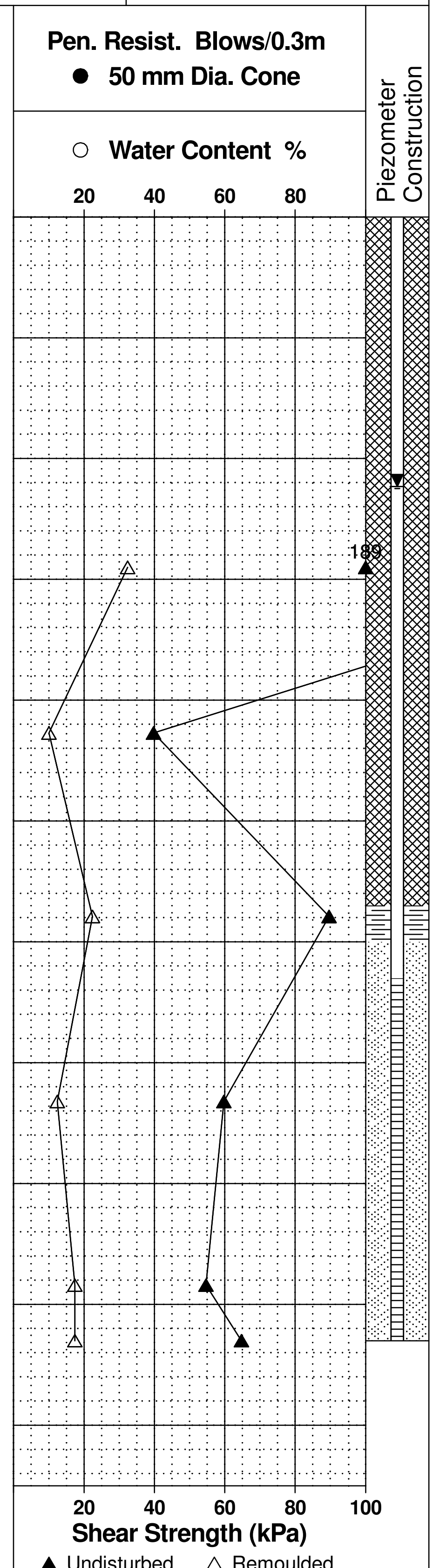
DATE 23 January 2019

FILE NO. PG4799

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05	SS	1	83	63
FILL: Crushed stone	0.20				
FILL: Brown sand, some silt and gravel	0.60	SS	2	83	5
Loose, brown **SILTY SAND,** some clay	1.30	SS	3	100	4
Very stiff to stiff, brown **SILTY CLAY**		SS	4	100	4
- grey by 3.8m depth		SS	5	100	W
		SS	6	100	W
		SS	7	100	W
	9.30	SS	8	100	W
Dynamic Cone Penetration Test commenced at 9.30m depth, cone pushed to 13.3m depth.					

DEPTH (m)	ELEV. (m)
0	71.95
1	70.95
2	69.95
3	68.95
4	67.95
5	66.95
6	65.95
7	64.95
8	63.95
9	62.95
10	61.95



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
347 Gilmour Street, 278, 280 & 282 O'Connor Street
Ottawa, Ontario

DATUM TBM - Top of grate of manhole cover located on Gilmour Street, near the southeast corner of subject site. Geodetic elevation = 71.16m.

REMARKS

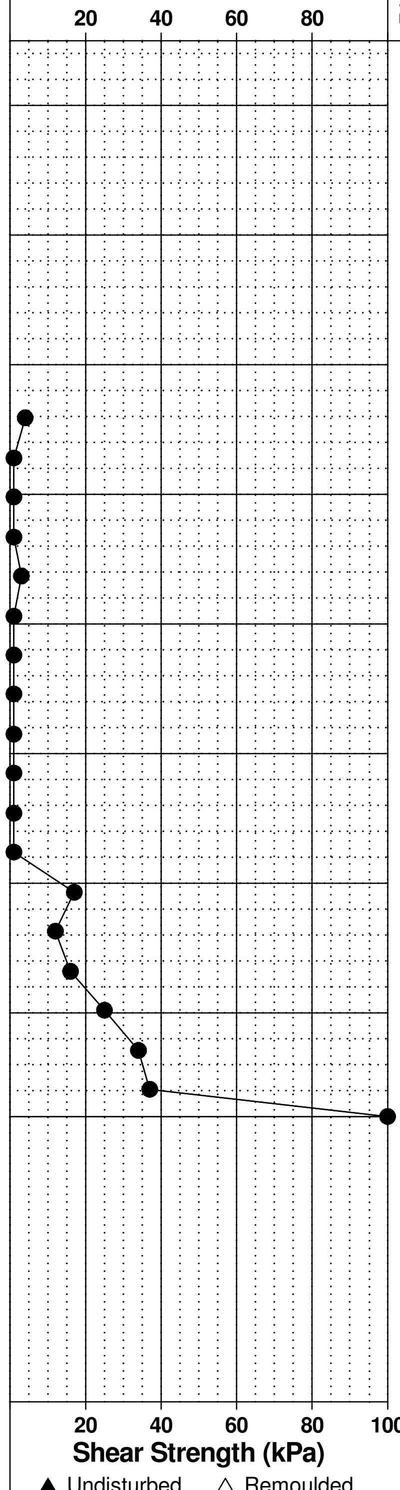
BORINGS BY Geoprobe

DATE 23 January 2019

FILE NO. PG4799

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE									
						11	60.95		
						12	59.95		
						13	58.95		
						14	57.95		
						15	56.95		
						16	55.95		
						17	54.95		
						18	53.95		
18.80									
End of Borehole Practical DCPT refusal at 18.80m depth (GWL @ 2.23m - Jan. 31, 2019)									



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
347 Gilmour Street, 278, 280 & 282 O'Connor Street
Ottawa, Ontario

DATUM TBM - Top of grate of manhole cover located on Gilmour Street, near the southeast corner of subject site. Geodetic elevation = 71.16m.

REMARKS

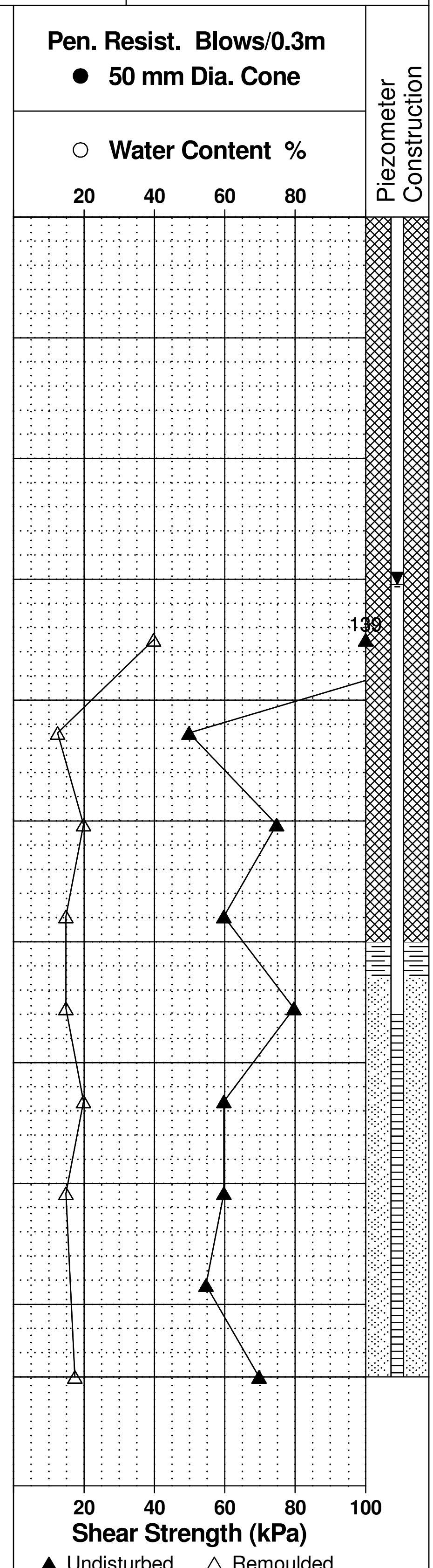
BORINGS BY CME 55 Power Auger

DATE 23 January 2019

FILE NO. PG4799

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	71.94
Asphaltic concrete 0.08							
FILL: Sand with gravel 0.30							
FILL: Brown sand with silt and gravel 1.52		SS	1	50	10	1	70.94
Very stiff to stiff, brown **SILTY CLAY**		SS	2	100	7	2	69.94
		SS	3	100	4		
						3	68.94
						4	67.94
- grey by 4.6m depth						5	66.94
						6	65.94
						7	64.94
						8	63.94
9.60						9	62.94
End of Borehole							
(GWL @ 3.04m - Jan. 31, 2019)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
347 Gilmour Street, 278, 280 & 282 O'Connor Street
Ottawa, Ontario

DATUM TBM - Top of grate of manhole cover located on Gilmour Street, near the southeast corner of subject site. Geodetic elevation = 71.16m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 23 January 2019

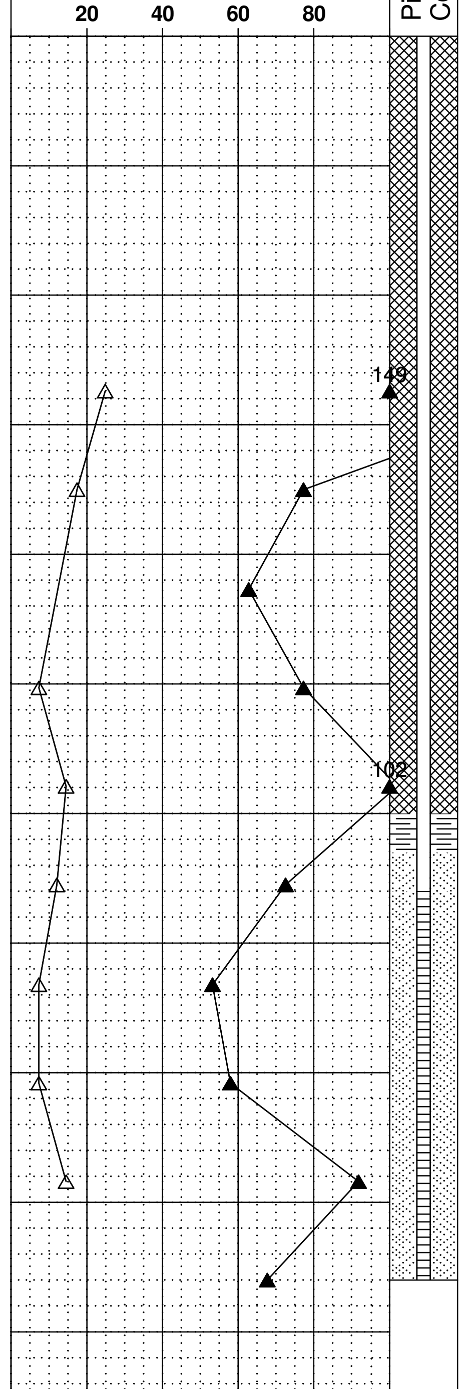
FILE NO. PG4799

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	71.74
Asphaltic concrete (0.05)							
FILL: Sand and gravel (0.25)		AU	1				
FILL: Brown silty sand, some clay and gravel (1.27)		SS	2	100	11	1	70.74
Very stiff to stiff, brown **SILTY CLAY**		SS	3	100	6	2	69.74
						3	68.74
						4	67.74
						5	66.74
- grey by 5.3m depth						6	65.74
						7	64.74
						8	63.74
(9.60)						9	62.74
Dynamic Cone Penetration Test commenced at 9.60m depth, cone pushed to 15.3m depth.						10	61.74

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

○ Water Content %



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers 154 Colonnade Road South, Ottawa, Ontario K2E 7J5	SOIL PROFILE AND TEST DATA Geotechnical Investigation 347 Gilmour Street, 278, 280 & 282 O'Connor Street Ottawa, Ontario

DATUM TBM - Top of grate of manhole cover located on Gilmour Street, near the southeast corner of subject site. Geodetic elevation = 71.16m.

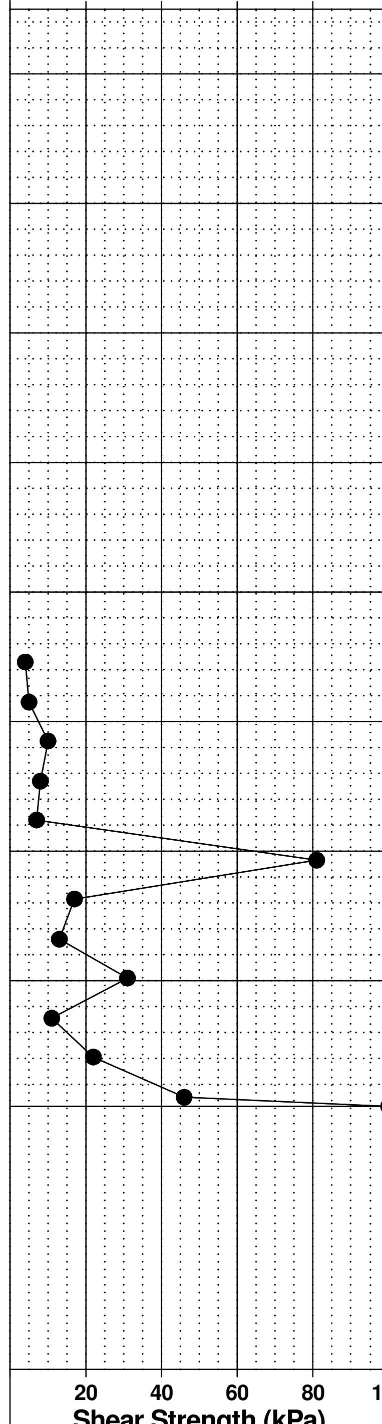
REMARKS

BORINGS BY CME 55 Power Auger **DATE** 23 January 2019

FILE NO. PG4799

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE									
						11	60.74		
						12	59.74		
						13	58.74		
						14	57.74		
						15	56.74		
						16	55.74		
						17	54.74		
						18	53.74		
18.97									
End of Borehole Practical DCPT refusal at 18.97m depth. (Piezometer blocked - Jan. 31, 2019)									



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic Limit, % (water content above which soil behaves plastically)
PI	-	Plasticity Index, % (difference between LL and PL)
Dxx	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

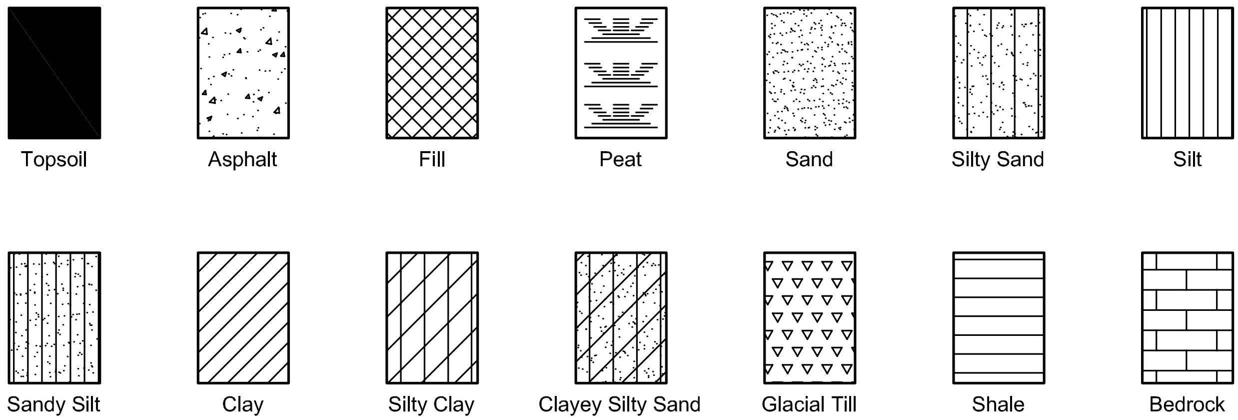
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

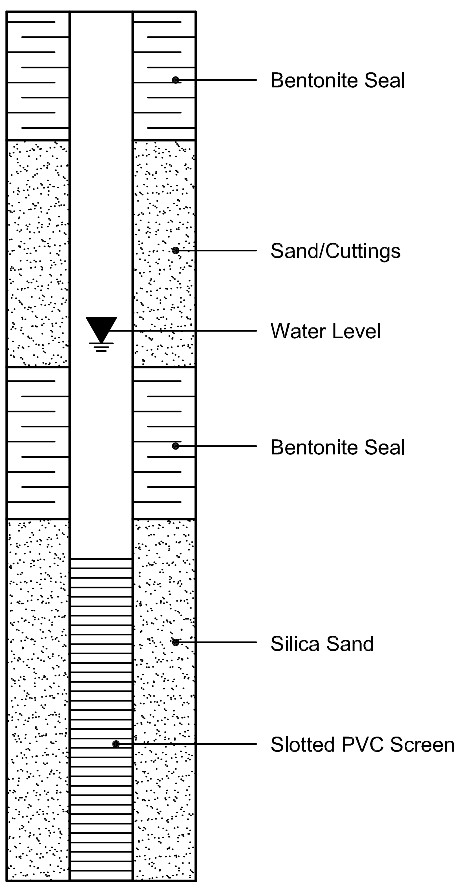
SYMBOLS AND TERMS (continued)

STRATA PLOT

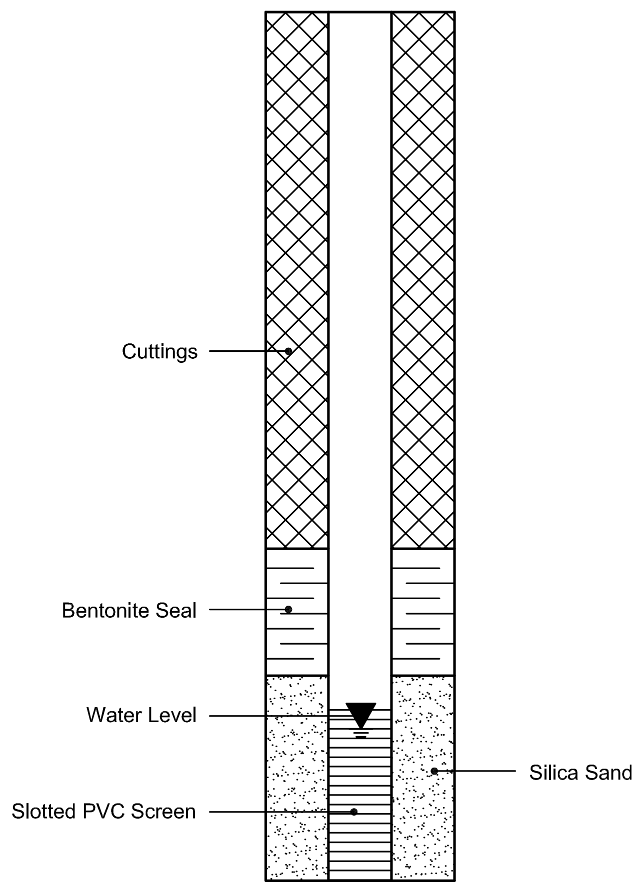


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

Order #: 1904487

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 24615

Report Date: 30-Jan-2019

Order Date: 25-Jan-2019

Project Description: PG4799

	MDL/Units				
Client ID:		BH1-SS4	-	-	-
Sample Date:		01/24/2019 09:00	-	-	-
Sample ID:		1904487-01	-	-	-
		Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	65.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.42	-	-	-
Resistivity	0.10 Ohm.m	22.2	-	-	-
Anions					
Chloride	5 ug/g dry	148	-	-	-
Sulphate	5 ug/g dry	80	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4799-1 - TEST HOLE LOCATION PLAN

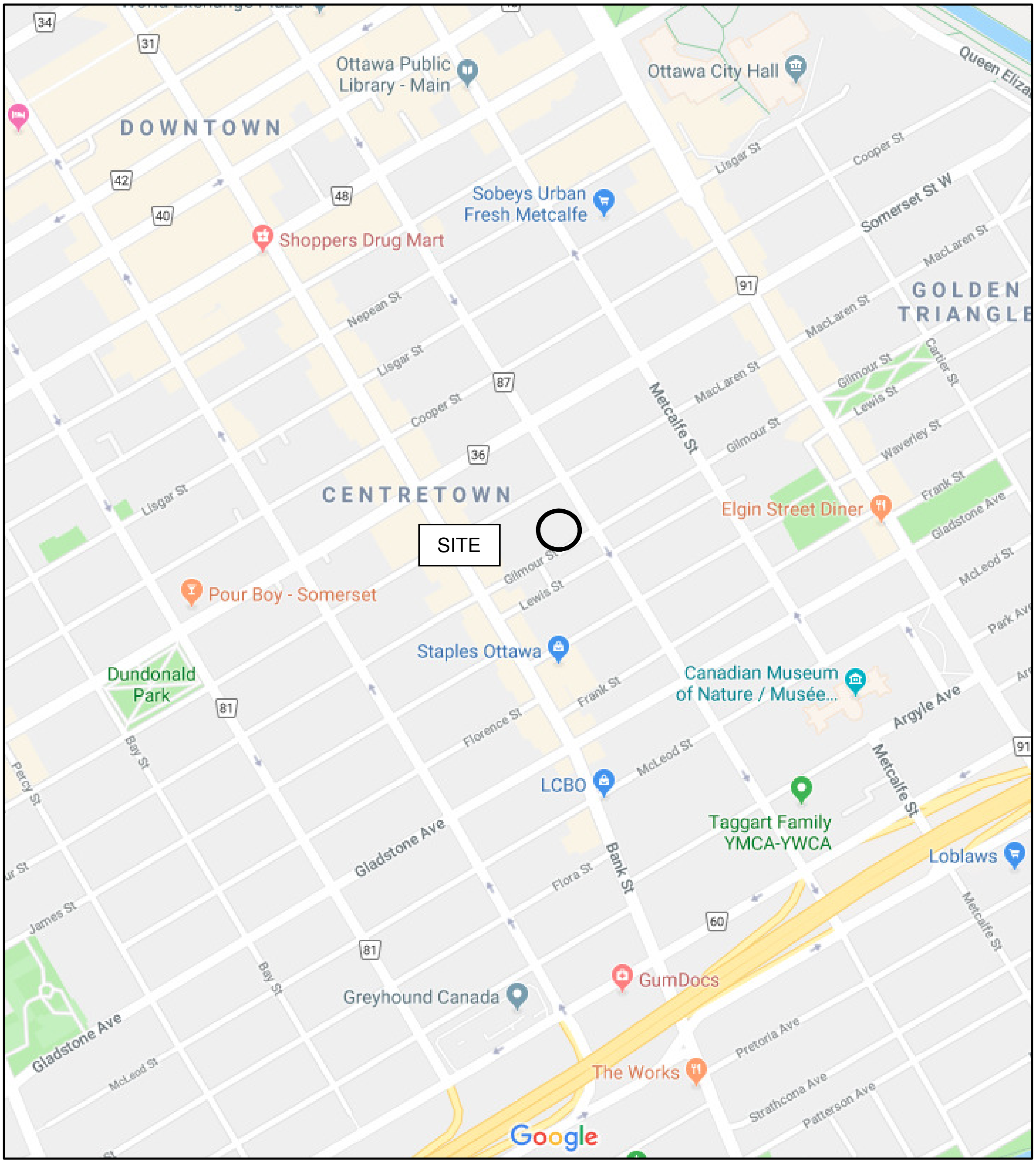
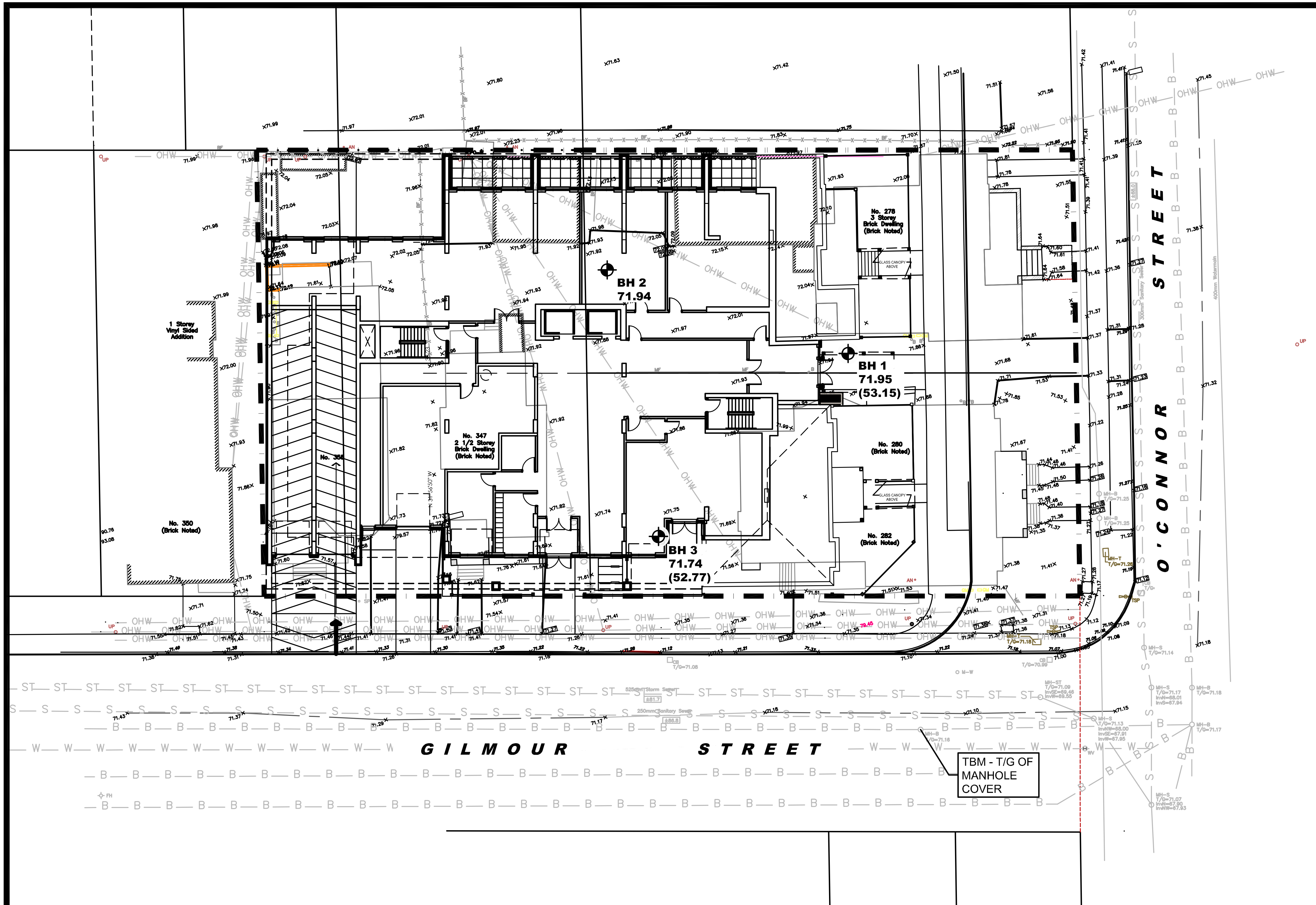


FIGURE 1

KEY PLAN



LEGEND:

⌖	BOREHOLE LOCATION
71.95	GROUND SURFACE ELEVATION (m)
(53.15)	PRACTICAL DCPT REFUSAL ELEV. (m)

TBM - TOP OF GRATE OF MANHOLE. GEODETIC ELEVATION = 71.16m, AS PER TOPOGRAPHIC PLAN PROVIDED BY FARLEY, SMITH & DENIS SURVEYING LIMITED.

SCALE: 1:300

0 1 2 3 4 5 10 15 20m

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL

AK GLOBAL MANAGEMENT
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
347 GILMOUR ST., 278, 280 & 282 O'CONNOR STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:300	Date: 01/2019
Drawn by: MPG	Report No.: PG4799-1
Checked by: NC	PG4799-1
Approved by: DJG	Revision No.:

p:\autocad drawings\geotechnical\pg47xx\pg4799-1 thlp.dwg