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Geotechnical Investigation

Proposed Athletic Recreation Complex (ARC)
Algonquin College
Woodroffe Campus - Ottawa

Prepared For

Algonquin College
c/o Colliers Project Leaders

June 3, 2019

Report: PG4624-1 Revision 1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Colliers Project Leaders on behalf of Algonquin College to conduct a geotechnical investigation for the proposed Athletic Recreation Complex (ARC) to be constructed at the Algonquin College Woodroffe Campus in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the geotechnical investigation was to:

❏ determine the subsurface soil and groundwater conditions by means of boreholes.

❏ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. The report contains the geotechnical findings and includes recommendations pertaining to the design and construction of the subject development as 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.

2.0 Proposed Development

For the proposed Athletic Recreation Complex (ARC), it's our understanding that the proposed development will consist of a one-storey slab-on-grade building with a partial basement to be used as gymnasium and other facilities. The proposed development will occupy the majority of the existing parking area. It is also expected that the proposed building will be fully municipally serviced and will be integrated with the existing surrounding hard surfaces.

Furthermore, it's our understanding that an underground storm water storage system will be installed south of the proposed building footprint. A hydrogeological review of the infiltration potential will be presented in a separate report.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on August 16, 2018. At that time, a total of 9 boreholes were drilled to a maximum depth of 6.7 m below existing ground surface. The test hole locations were determined in the field by Paterson personnel and distributed in a manner to provided general coverage of the proposed project known at the time of the field portion of the geotechnical investigation while taking into consideration of site features and underground utilities. The test hole locations are presented on Drawing PG4624-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a truck-mounted auger drill 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 test hole procedures consisted of augering to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

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

Standard Penetration Testing (SPT) was conducted and 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 sample 300 mm into the soil after a 150 mm initial penetration with a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing was conducted at regular intervals in cohesive soils and completed using a MTO field vane apparatus.

Overburden thickness was evaluated by a dynamic cone penetration test (DCPT) at BH 2, BH 3 and BH 4. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip and 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 every 300 mm.

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 standpipes were installed in the boreholes during the field investigation to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The test holes completed during the field investigation were selected in the field and surveyed by Paterson. The ground surface elevations at the test hole locations were referenced to a geodetic benchmark, consisting of the top of spindle of the fire hydrant located to the south of the subject section of the site. An geodetic elevation of 86.84 m was assigned to this benchmark.

The location of the TBM, test hole locations and ground surface elevation at each test hole location are presented on Drawing PG4624-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

One 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 majority of the subject site is currently occupied by an at-grade, asphalt covered car parking and access lanes. A landscaped area with mature trees was also noted along the north boundary of the site. The subject site is relatively flat with a slight downslope towards the northwest portion.

Based on historical aerial photographs of the site, a former drainage ditch bisected the west portion of the subject site at the approximate location of the proposed Athletic Recreation Complex (ARC). The former drainage ditch ran in a north-south direction across the west portion of the subject section of the site. The aerial photograph from 1965 illustrated in Figure 2 in Appendix 2 identifies the approximate alignment of the former drainage ditch. The approximate location of the former drainage ditch has been further presented in an aerial photograph of 1991 and 2017 in Figure 3 and Figure 4, respectively.

The site is bordered to the north by a 3 storey Algonquin Commons Theater, to the east and south by an at-grade asphalt covered parking area.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the test hole locations consists a pavement structure overlying a hard to stiff brown silty clay crust followed by a very stiff to stiff grey silty clay deposit. Glacial till was encountered at BH 4 consisting of grey silty clay with sand and gravel. It should be noted that a fill layer consisting of brown silty sand with crushed stone and/or brown silty clay with sand and gravel was encountered within BH 1, BH 2 and BH 4 where the former drainage ditch ran along the west portion of the site. In addition, a layer of topsoil and organics was encountered directly below the fill material at BH 1 and BH 2.

Practical refusal to DCPT was encountered at a depth of 9.9, 8.5 and 9.1m at BH 2, BH 3 and BH 4, respectively.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

4.3 Groundwater

The measured groundwater levels are summarized below in Table 1 and presented on the Soil Profile and Test Data sheets in Appendix 1. It should be noted that surface water can become perched with a backfilled borehole, which can lead to higher than normal groundwater level readings. The long-term groundwater level can also be estimated based on the recovered soil samples' moisture levels, colouring and consistency. Based on these observations, the long-term groundwater level is anticipated at a **4 to 5 m depth**. Groundwater levels are subject to seasonal fluctuations and could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Date
BH 1	85.84	3.08	82.76	August 21, 2018
BH 2	86.07	3.07	83.00	August 21, 2018
BH 3	86.55	2.82	83.73	August 21, 2018
BH 4	85.58	2.96	82.62	August 21, 2018
BH 5	86.24	2.84	83.40	August 21, 2018
BH 6	86.92	Blocked	-	August 21, 2018
BH 7	86.82	Blocked	-	August 21, 2018
BH 8	86.67	2.04	84.63	August 21, 2018
BH 9	87.08	2.72	84.36	August 21, 2018

Note: The ground surface at the test hole locations was referenced to a geodetic benchmark consisting of the top of spindle of the fire hydrant located to the south of the subject section of the site. A elevation of 86.84 was assigned to the benchmark.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is satisfactory for the proposed development. It’s expected that the proposed building will be founded on conventional spread footings placed on an undisturbed, stiff silty clay bearing surface.

Where the existing fill is encountered at design underside of footing elevation, it’s expected that the footings will be either extended to reach an undisturbed, silty clay bearing surface, placed on an approved engineered fill or placed on lean concrete in-filled trench that extends to an undisturbed, silty clay bearing surface.

Consideration could be given to leaving the existing fill under the proposed slab-on-grade. It is recommended to sub-excavate an additional 300 mm below the proposed subgrade where existing fill is currently present. The fill subgrade should be proof rolled, under dry conditions, making several passes and approved by the geotechnical consultant at the time of construction. The sub-excavated area should then be in-filled with OPSS Granular B Type II and compacted to a minimum 98% of the material’s SPMDD up to the underside of footing elevation.

Due to the presence of the silty clay layer, grading in close proximity to any settlement sensitive structures will be subjected to a permissible grade raise restriction.

To ensure that the proposed basement area remains dry and prone to less moisture intrusion, a water suppression system is recommended to manage and reduce the volume of water infiltration over the long term at post-construction.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. It’s expected that the existing fill, free of deleterious material and significant amounts of organics, can be left in place below the proposed building footprint, outside of lateral support zones for the footings, and below the proposed parking area and access lane. However, it is recommended that the existing fill layer be proof-rolled several times and approved by the geotechnical consultant at the time of construction.

Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill. Existing foundation walls and/or other construction debris, where present, should be entirely removed from within the building perimeter.

Fill Placement

Fill placed 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 maximum 300 mm thick lifts and compacted to 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts compacted by the tracks of the spreading equipment to minimize voids. If the material is to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm lifts and compacted to 95% of the material's SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement 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 (Slab-on-Grade Portion)

Pad footings, up to 6 m wide, and strip footings, up to 3 m wide, founded on an undisturbed, stiff silty clay bearing surface or over an approved engineered fill extending to an undisturbed, silty clay bearing surface can be designed using the bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 kPa**. Alternatively, footings founded over a lean concrete in-filled trench as detailed below can be designed using the abovenoted SLS and ULS values.

A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

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

Bearing Resistance Values (Basement)

For the basement portion of the building within the western portion, it's expected that the depth of the foundation will be approximately 3 m below the existing grade. Minimal to no sub-excavation of fill will be required to be removed below the proposed underside of footings due to the depth of the founding level.

To protect the soil bearing surface during construction and to create a horizontal hydraulic barrier at depth, it's recommended that a concrete mud slab be placed immediately after exposure of the bearing surface. The bearing surface should be inspected by the geotechnical engineer prior to concrete placement. The concrete mud slab should consist of a 150 mm thick layer with a minimum 25 MPa compressive strength concrete.

Footings placed on 25 MPa concrete mud slab overlying an undisturbed, stiff silty clay bearing surface or over an approved engineered fill extending to an undisturbed, silty clay bearing surface can be designed using the bearing resistance value at serviceability limit states (SLS) of **175 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**.

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

The above noted bearing resistance value at SLS will be subjected to 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 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

A permissible grade raise restriction of **2 m** above existing ground surface is recommended for the proposed building.

5.4 Design for Earthquakes

The subject site can be taken as seismic site response **Class C** as defined in the Ontario Building Code 2012 (OBC 2012; Table 4.1.8.4.A) for foundations considered at this site. The soils underlying the site are not susceptible to liquefaction.

5.5 Slab on Grade/Basement Slab Construction

With the removal of all topsoil and deleterious fill, such as those containing significant amounts of organic matter, within the footprint of the proposed building, undisturbed native soil surface or existing fill, approved by the geotechnical consultant at the time of construction, will be considered acceptable subgrade on which to commence backfilling for floor slab construction. It's recommended that the existing fill layer, free of deleterious and organic materials, be proof-rolled (if possible due to moisture content) and approved by the geotechnical consultant at the time of construction. Any soft areas should be removed and backfilled with suitable dryer backfill material. It's recommended that the upper 200 mm of sub-slab fill of a slab-on-grade construction to consist of an OPSS Granular A crushed stone.

For the basement slab, it's our understanding that the floor will be heated and will have an insulation layer. Furthermore, since a concrete mud slab will be used to create a hydraulic barrier, the material to backfill above the concrete mud slab will consist of a free draining material such as an OPSS Granular A and/or possibly a layer of 19 mm clear crushed stone. Subfloor drainage will be incorporated in this design which will permit the subfloor material to remain dry.

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 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.

Lateral 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)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire wall height should be incorporated to the 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 calculated 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 stay at least 0.3 m away 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}) could 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. 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 following pavement structures presented below could be used for the design of car parking areas, bus lanes and access lanes. It is anticipated that both pavement structures provided would be adequate for use as a fire route.

Table 2 - 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 3 - Recommended Pavement Structure
Bus Turning Areas and Access Lanes

Thickness (mm)	Material Description
40	**Wear Course -** Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil	

Minimum Performance Graded (PG) 64-34 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, 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 SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is dependent on the moisture condition of the contact zone between the subgrade material and granular base. Failure to provide adequate drainage under conditions of heavy wheel loading could result in the subgrade fines being pumped into the stone subbase voids, thereby reducing the load bearing capacity.

Due to the impervious nature of the subgrade materials consideration should be provided to installing subdrains during the pavement construction. The subdrains should extend in four orthogonal directions and 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 and placed in accordance with City of Ottawa standard drawings. 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

Water Suppression System and Foundation Drainage

To manage and control groundwater water infiltration over the long term, the following water suppression system is recommended to be installed for the exterior foundation walls and underfloor drainage (refer to Figure 5 - Water Suppression System in Appendix 2 for an illustration of this system cross-section):

☐ The concrete mud slab will create a horizontal hydraulic barrier to lessen the water infiltration at the base of the excavation and will consist of a 150 mm thick layer of 25 MPa compressive strength concrete. The 150 mm minimum thickness is required to enable the support of construction traffic until the footings are poured and the area is backfilled and minimize long term cracking.

☐ A composite drainage layer will be placed from finished grade to the bottom of the foundation wall. It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the bottom of the foundation wall. It’s expected that 150 mm diameter sleeves placed at 3 m centres be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to the sump pit(s) within the lower basement area.

☐ A waterproofing membrane will be required to lessen the effect of water infiltration for the basement level starting at 1.5 m below finished grade. The waterproofing membrane will consist of bentonite panels fastened to the composite drainage layer. The bentonite membrane should extend to the bottom of the excavation at the founding level of the proposed footings over the concrete mud slab.

☐ A sump pit should be designed to manage any groundwater infiltration which would be discharged to the sewer system. The sump pump should be designed to handle a maximum water volume of 200,000 L/day. However, once steady state is achieved, it’s expected that water infiltration volumes will be less than 5,000 L/day.

Underfloor Drainage

Underfloor drainage may be required to control water infiltration below the basement slab that breaches the horizontal hydraulic barrier (minimum 150 mm thick concrete mud slab). For design purposes, it's recommended that a 150 mm diameter perforated pipe be placed in each bay. The final spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Water Infiltration Volumes

During the construction phase, it's expected that water infiltration should have a steady state volume between 10,000 and 25,000 L/day plus any surface water infiltration following a precipitation event. The initial influx will be greater once the excavation extends below the long term groundwater level. The zone of influence associated with the temporary dewatering during construction excavation for 1 basement level will be approximately 5 m.

Based on the proposed water suppression system, it's expected that long term groundwater infiltration will be significantly reduced during post-construction. With a properly implemented water suppression system, it's expected that post-construction volumes will be less than 5,000 L/day.

6.2 Protection 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.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection. The recommended minimum thickness of soil cover is 2.1 m (or equivalent).

6.3 Excavation Side Slopes

The excavations for the proposed development will be through a native silty clay material. 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. Above the groundwater level, for excavations to depths of approximately 3 m, the excavation side slopes should be stable in the short term at 1H:1V. Shallower slopes should be provided for deeper excavations or for excavation below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring should be installed.

The slope cross-sections recommended above are for temporary slopes. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be maintain safe working distance from the excavation sides.

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

A trench box is recommended to be installed 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 should not remain open for extended periods of time.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with City of Ottawa standards and specifications.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extend at a minimum to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A crushed stone, should extend from the spring line of the pipe to a minimum of 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

Generally, the dry brown silty clay could be place above the cover material if the excavation and backfilling operations are completed in dry weather conditions. The wet silty clay materials could be difficult to place and compact, due to the high water content.

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 consist of 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 SPMDD.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be 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.

A temporary Ministry of the Environment, Conservation and Parks (MECP) Category 3 permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum of 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

Precautions should be provided if winter construction is considered for this project. The subsurface soil conditions 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.

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

The trench excavations should be constructed to avoid the introduction of frozen materials, snow or ice 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 during construction. Also, 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.

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

The following is recommended to be completed once the site plan and development are determined:

☐ Review detailed grading plan(s) from a geotechnical perspective.

☐ Observation and inspection of the water suppression system installation.

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

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to ensure that the specified level of compaction has been achieved.

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

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

A report confirming the construction has been completed in general accordance with the 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 report recommendations are in accordance with the present understanding of the project. Paterson requests permission to review the grading plan, once available, and recommendations when the drawings and specifications are complete.

The recommendations are based on information gathered at the specific test locations and could 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.

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 Algonquin College, Colliers Project Leaders 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.

Faisal I. Abou-Seido, P.Eng.

LICENSED PROFESSIONAL ENGINEER
June 3, 2019
C. P. DA SILVA
PROVINCE OF ONTARIO

Carlos P. Da Silva, P.Eng., ing., QP$_{ESA}$.

Report Distribution

☐ Colliers Project Leaders (3 copies)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Athletics Recreation Complex (ARC)
Algonquin College - Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located to the south of the subject section of site. Geodetic elevation = 86.84m.

REMARKS

BORINGS BY CME 55 Power Auger

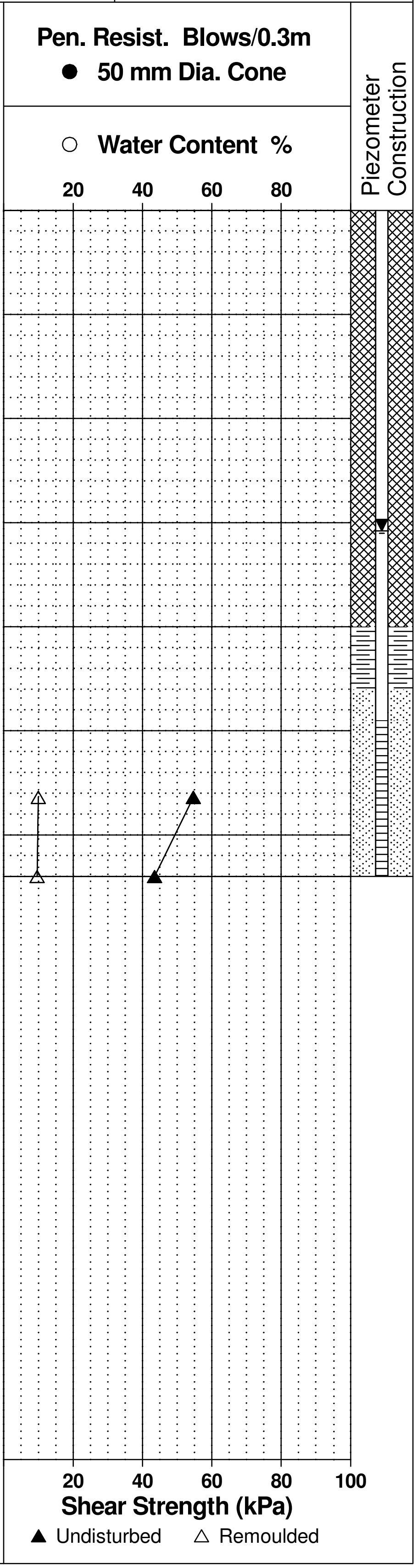
DATE 2018 August 16

FILE NO. PG4624

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT (depth, m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05	AU	1		
FILL: Brown silty sand with crushed stone and gravel	1.52	SS	2	67	6
FILL: Brown silty clay, some sand, trace gravel	3.73	SS	3	33	4
		SS	4	58	8
		SS	5	58	4
TOPSOIL and organics with silty clay	4.01	SS	6	75	8
Very stiff to stiff, brown **SILTY CLAY**	6.40	SS	7	100	0
End of Borehole					
(GWL @ 3.08m - August 21, 2018)					

DEPTH (m)	ELEV. (m)
0	85.84
1	84.84
2	83.84
3	82.84
4	81.84
5	80.84
6	79.84



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Athletics Recreation Complex (ARC)
Algonquin College - Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located to the south of the subject section of site. Geodetic elevation = 86.84m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2018 August 16

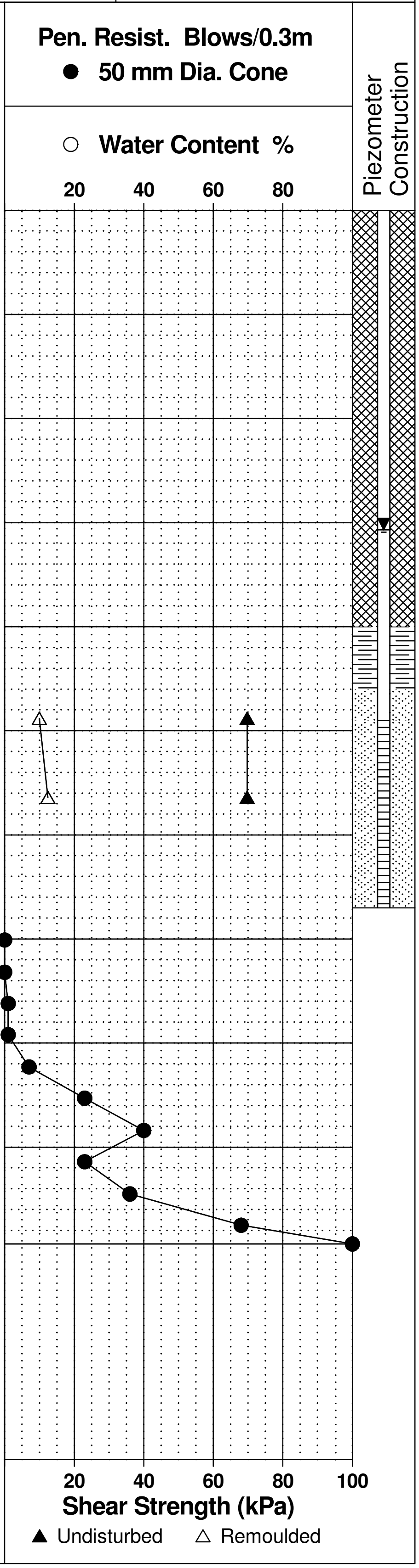
FILE NO. PG4624

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Brown silty sand with crushed stone and gravel	0.56
FILL: Brown silty clay, some sand, trace gravel	3.66
TOPSOIL and organics	4.01
Stiff, brown **SILTY CLAY**	5.33
Stiff, grey **SILTY CLAY,** trace sand	6.71
Dynamic Cone Penetration Test commenced at 6.71 m depth	9.93

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	63	8
SS	3	79	5
SS	4	58	6
SS	5	67	3
SS	6	54	4
SS	7	100	0

DEPTH (m)	ELEV. (m)
0	86.07
1	85.07
2	84.07
3	83.07
4	82.07
5	81.07
6	80.07
7	79.07
8	78.07
9	77.07



End of Borehole

Practical refusal to DCPT @ 9.93 m depth

(GWL @ 3.07m - August 21, 2018)

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Athletics Recreation Complex (ARC)
Algonquin College - Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located to the south of the subject section of site. Geodetic elevation = 86.84m.

REMARKS

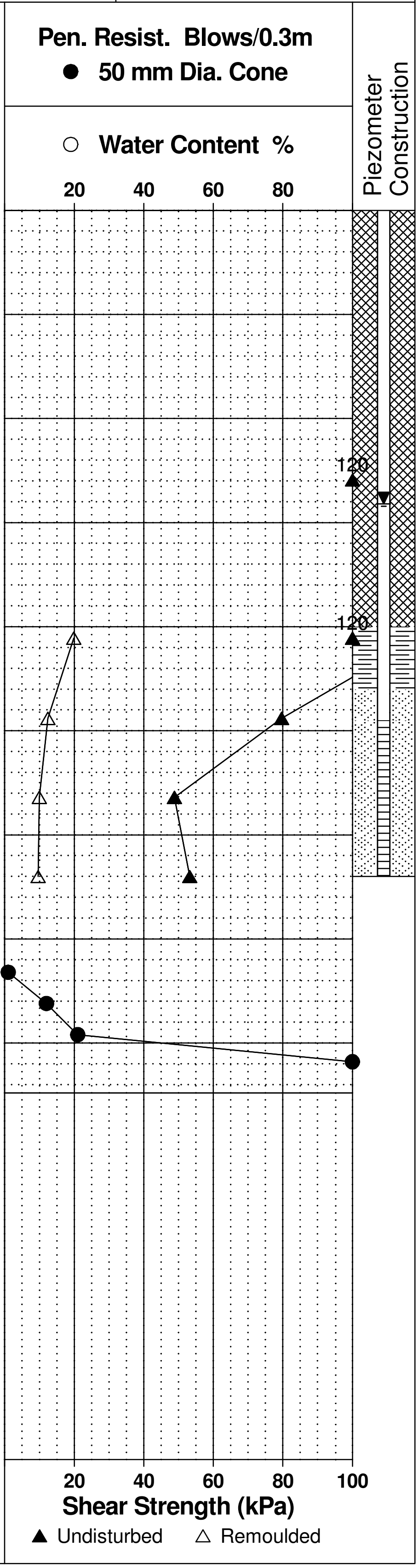
BORINGS BY CME 55 Power Auger

DATE 2018 August 16

FILE NO. PG4624

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.55
Asphaltic concrete 0.05		AU	1				
FILL: Crushed stone with brown silty sand, some gravel 0.53							
FILL: Compact, brown silty sand 1.52		SS	2	58	10	1	85.55
Very stiff to stiff, brown **SILTY CLAY** 5.33		SS SS	3 4	100 100	6 5	2 3 4 5	84.55 83.55 82.55 81.55
Very stiff to stiff, brown **SILTY CLAY**, trace gravel -trace sand and gravel by 6.1 m depth 6.40						6	80.55
Dynamic Cone Penetration Test commenced at 6.40 m depth 8.48						7 8	79.55 78.55
End of Borehole Practical refusal to DCPT @ 8.48 m depth (GWL @ 2.82m - August 21, 2018)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Athletics Recreation Complex (ARC)
Algonquin College - Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located to the south of the subject section of site. Geodetic elevation = 86.84m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2018 August 16

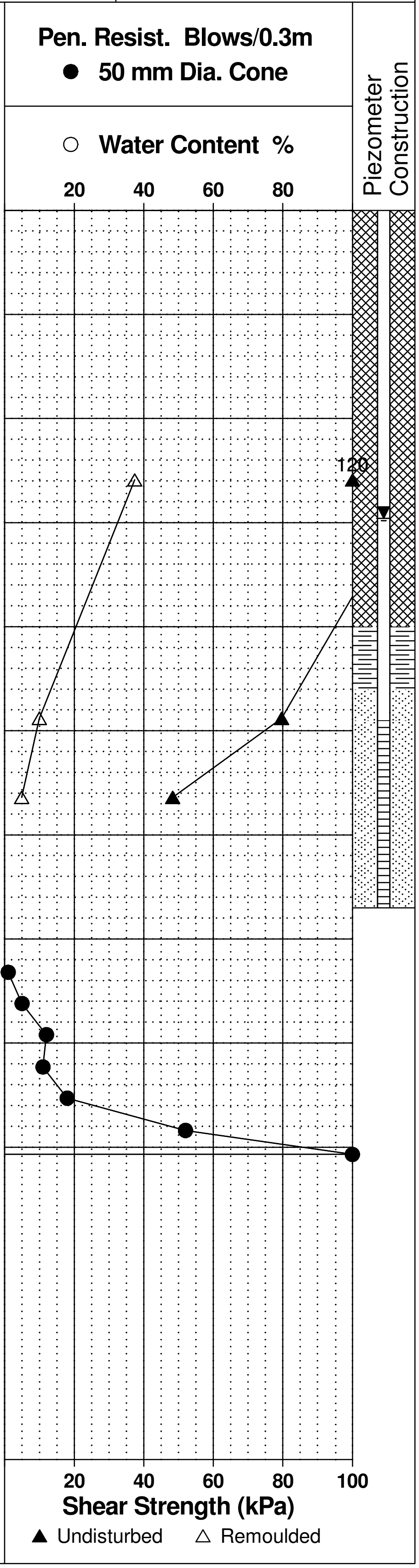
FILE NO. PG4624

HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.08
FILL: Brown silty sand with crushed stone and gravel	0.53
FILL: Brown silty clay with sand, some gravel	3.05
Very stiff to stiff, brown **SILTY CLAY** - grey by 5.3 m depth	6.10
GLACIAL TILL: Stiff, grey silty clay with sand, some gravel	6.71
Dynamic Cone Penetration Test commenced at 6.71m depth.	9.07
End of Borehole Practical refusal to DCPT @ 9.07 m depth (GWL @ 2.96m - August 21, 2018)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	6
SS	3	67	4
SS	4	88	6
SS	5	100	5
SS	6	100	3

DEPTH (m)	ELEV. (m)
0	85.58
1	84.58
2	83.58
3	82.58
4	81.58
5	80.58
6	79.58
7	78.58
8	77.58
9	76.58



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Athletics Recreation Complex (ARC)
Algonquin College - Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located to the south of the subject section of site. Geodetic elevation = 86.84m.

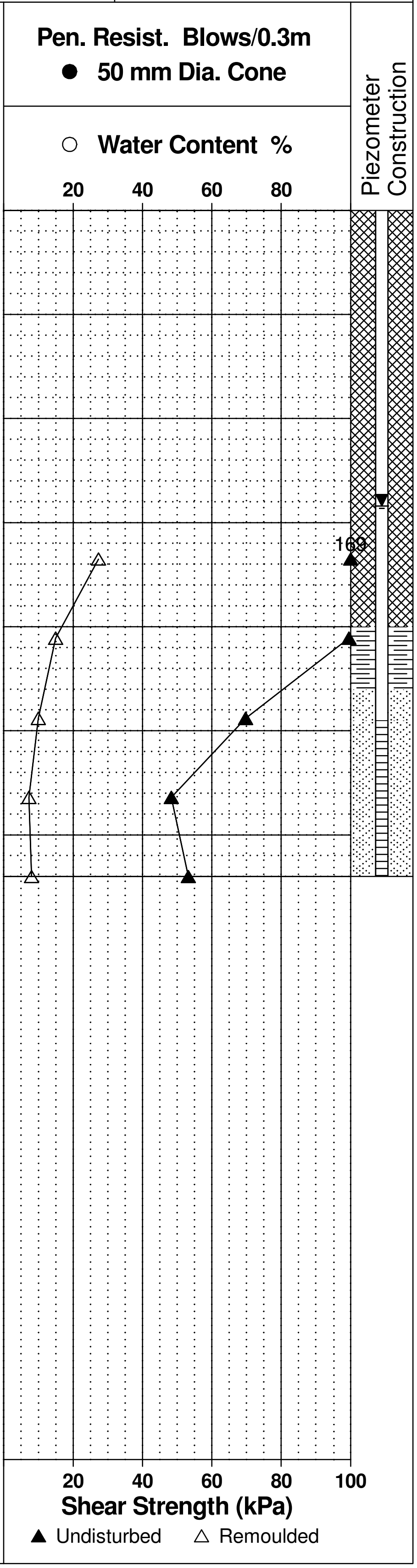
REMARKS

BORINGS BY CME 55 Power Auger **DATE** 2018 August 16

FILE NO. PG4624

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	86.24
Asphaltic concrete 0.04							
FILL: Brown silty sand with crushed stone and gravel 0.51		AU	1				
		SS	2	92	8	1	85.24
		SS	3	100	9	2	84.24
Very stiff to stiff, brown **SILTY CLAY**		SS	4	100	6		
						3	83.24
						4	82.24
						5	81.24
- grey by 5.3 m depth							
						6	80.24
6.40							
End of Borehole							
(GWL @ 2.84m - August 21, 2018)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Athletics Recreation Complex (ARC)
Algonquin College - Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located to the south of the subject section of site. Geodetic elevation = 86.84m.

REMARKS

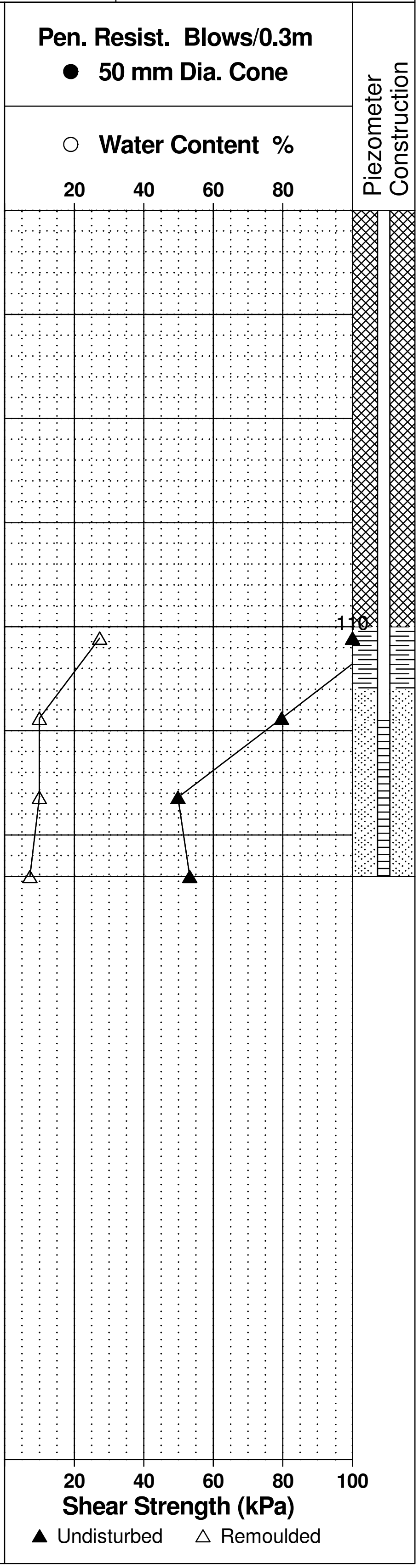
BORINGS BY CME 55 Power Auger

DATE 2018 August 16

FILE NO. PG4624

HOLE NO. BH 6

SOIL DESCRIPTION		SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE							
Asphaltic concrete	0.06	AU	1			0	86.92
FILL: Brown silty sand with crushed stone and gravel	0.56						
Hard to stiff, brown **SILTY CLAY**		SS	2	79	17	1	85.92
		SS	3	100	8	2	84.92
		SS	4	100	9	3	83.92
		SS	5	100	6	4	82.92
- grey by 5.3 m depth						5	81.92
	6.40					6	80.92
End of Borehole (Piezometer blocked at 3.05m depth - August 21, 2018)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Athletics Recreation Complex (ARC)
Algonquin College - Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located to the south of the subject section of site. Geodetic elevation = 86.84m.

REMARKS

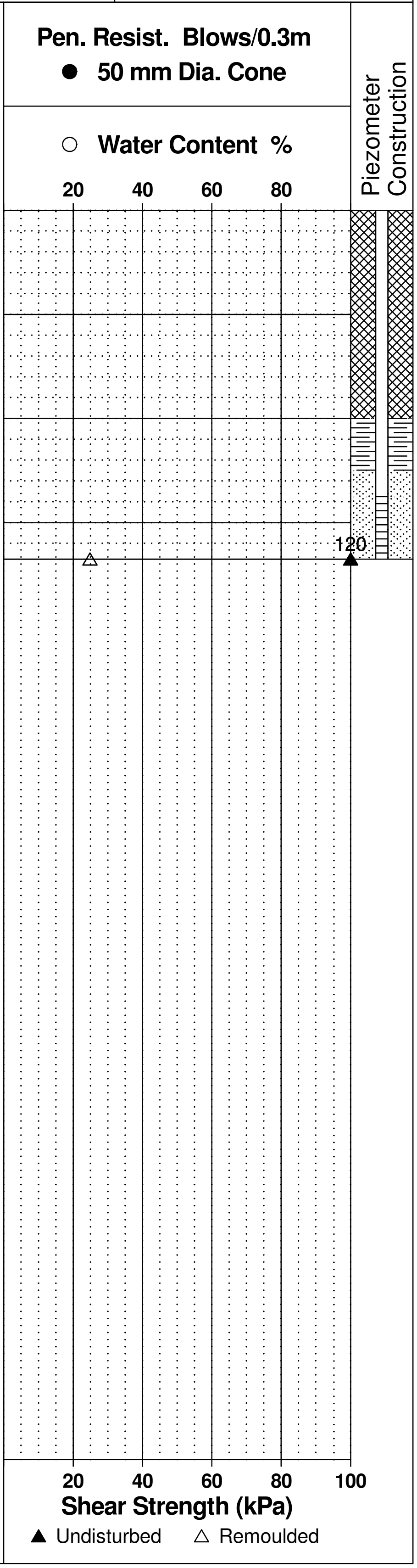
BORINGS BY CME 55 Power Auger

DATE 2018 August 16

FILE NO. PG4624

HOLE NO. BH 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.82
Asphaltic concrete 0.05							
FILL: Crushed stone, some sand, trace silty clay 0.53		AU	1				
Very stiff to stiff, brown **SILTY CLAY**		SS	2	79	6	1	85.82
						2	84.82
		SS	3	100	8		
						3	83.82
3.35		SS	4	100	5		
End of Borehole (Piezometer blocked @ 2.81m depth - August 21, 2018)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Athletics Recreation Complex (ARC)
Algonquin College - Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located to the south of the subject section of site. Geodetic elevation = 86.84m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2018 August 16

FILE NO. PG4624

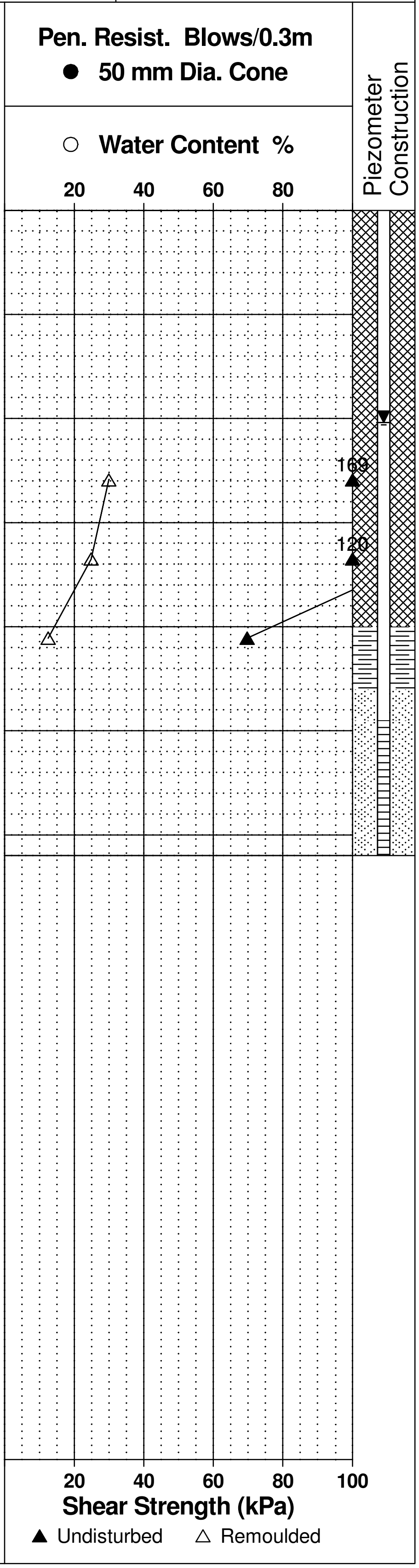
HOLE NO. BH 8

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.67
Asphaltic concrete 0.05							
FILL: Brown silty sand with crushed stone and gravel 0.53		AU	1				
		SS	2	100	9	1	85.67
						2	84.67
Very stiff to stiff, brown **SILTY CLAY**		SS	3	100	6		
						3	83.67
4.57						4	82.67
GLACIAL TILL: Hard to stiff, grey silty clay, some sand, gravel, cobbles and boulders 5.33		SS	4	100	50+	5	81.67
GLACIAL TILL: Dense, grey silty sand with gravel, occasional cobbles and boulders, some clay 6.20		SS	5	63	44	6	80.67
		SS	6	100	50+		

End of Borehole

Practical refusal to augering at 6.20m depth

(GWL @ 2.04m - August 21, 2018)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Athletics Recreation Complex (ARC)
Algonquin College - Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located to the south of the subject section of site. Geodetic elevation = 86.84m.

REMARKS

BORINGS BY CME 55 Power Auger

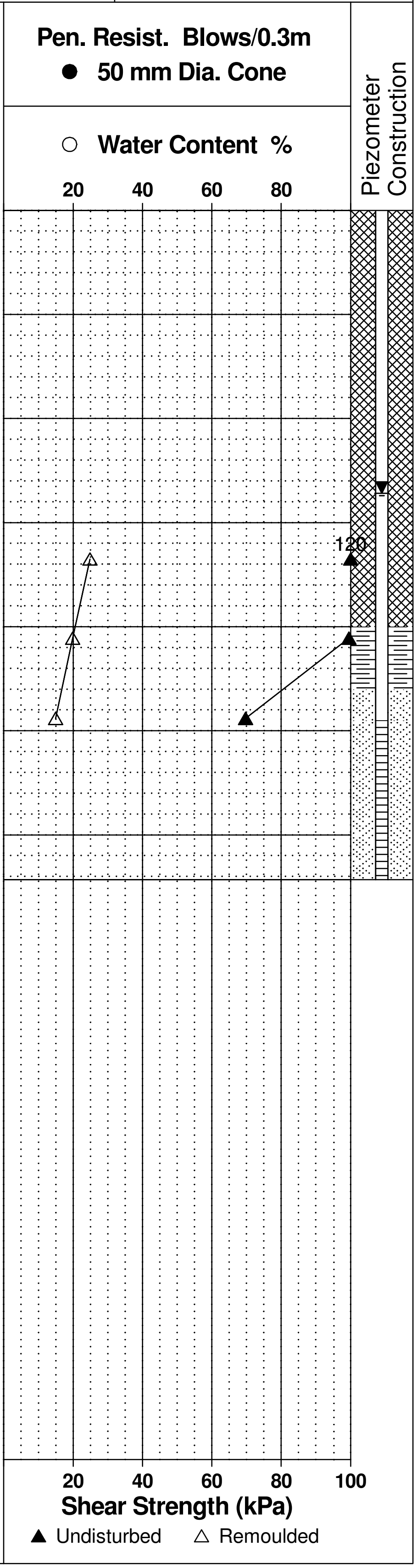
DATE 2018 August 16

FILE NO. PG4624

HOLE NO. BH 9

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE				
Asphaltic concrete 0.06	AU	1		
FILL: Brown silty sand with crushed stone and gravel	SS	2	54	28
1.52	SS	3	75	8
Very stiff to stiff, brown **SILTY CLAY**	SS	4	100	7
5.33				
GLACIAL TILL: Hard, brown silty clay, trace sand and gravel, occasional cobbles and boulders	SS	5	100	50+
6.43	SS	6	100	50+
End of Borehole Practical refusal to augering at 6.43m depth (GWL @ 2.72m - August 21, 2018)				

DEPTH (m)	ELEV. (m)
0	87.08
1	86.08
2	85.08
3	84.08
4	83.08
5	82.08
6	81.08



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)

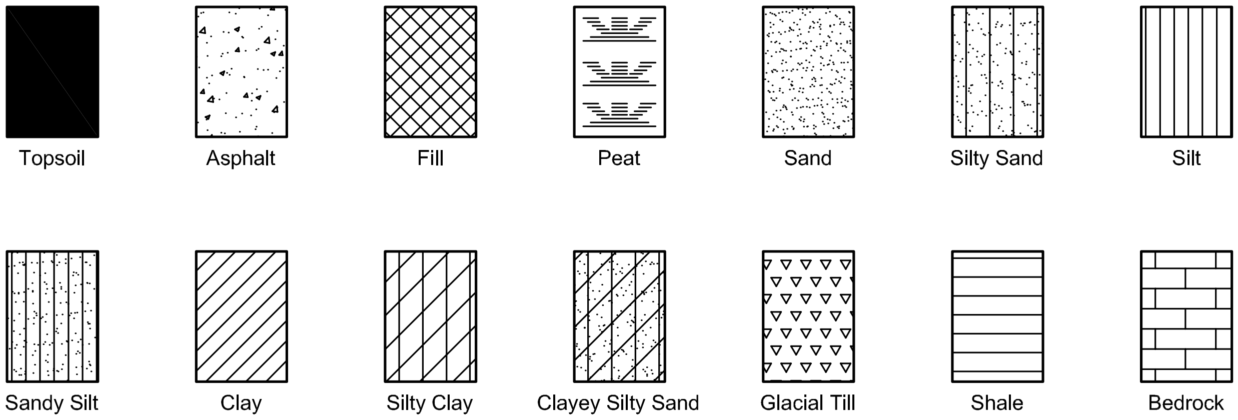
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

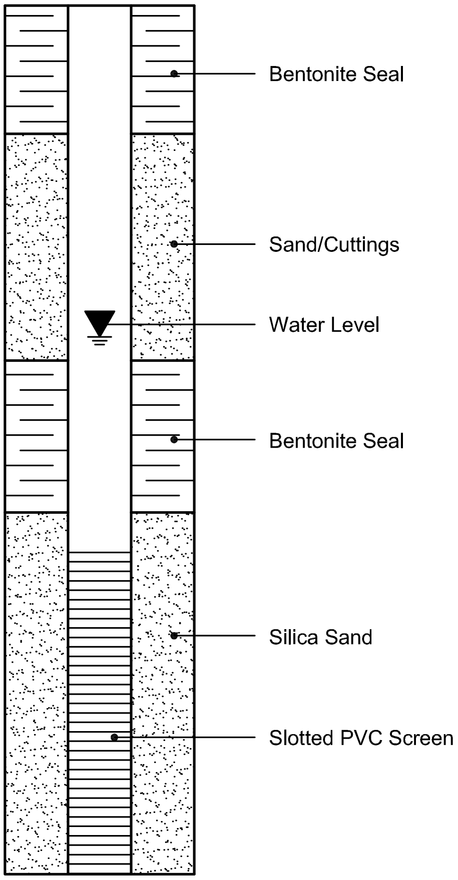
SYMBOLS AND TERMS (continued)

STRATA PLOT

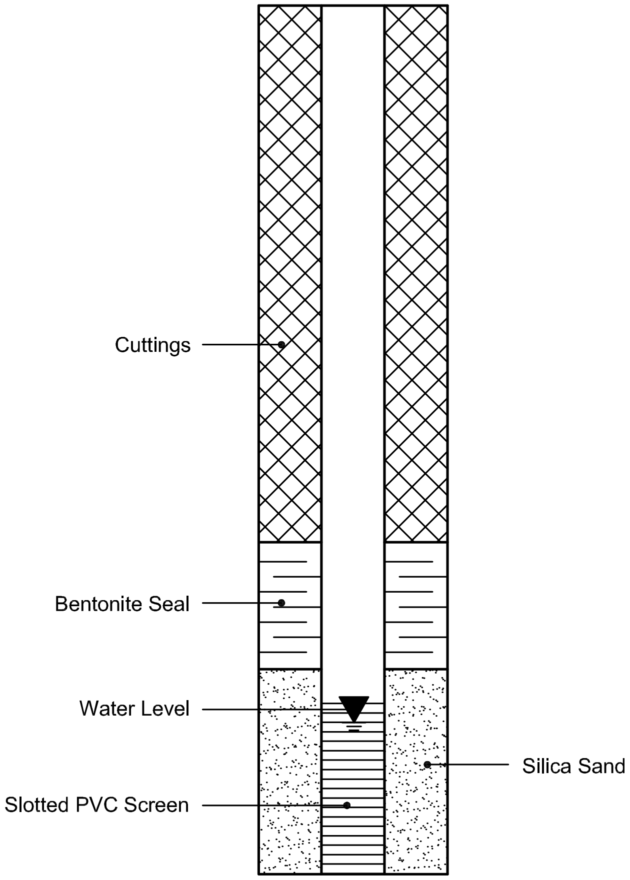


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 2491

Report Date: 27-Aug-2018

Order Date: 21-Aug-2018

Project Description: PG4624

	Client ID:	BH3-SS3	-	-	-
	Sample Date:	08/16/2018 15:30	-	-	-
	Sample ID:	1834311-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	73.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.60	-	-	-
Resistivity	0.10 Ohm.m	2.19	-	-	-
Anions					
Chloride	5 ug/g dry	3180	-	-	-
Sulphate	5 ug/g dry	344	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - AERIAL PHOTOGRAPH - 1965

FIGURE 3 - AERIAL PHOTOGRAPH - 1991

FIGURE 4 - AERIAL PHOTOGRAPH - 2017

FIGURE 5 - WATER SUPPRESSION SYSTEM DETAIL

DRAWING PG4624-1 - TEST HOLE LOCATION PLAN

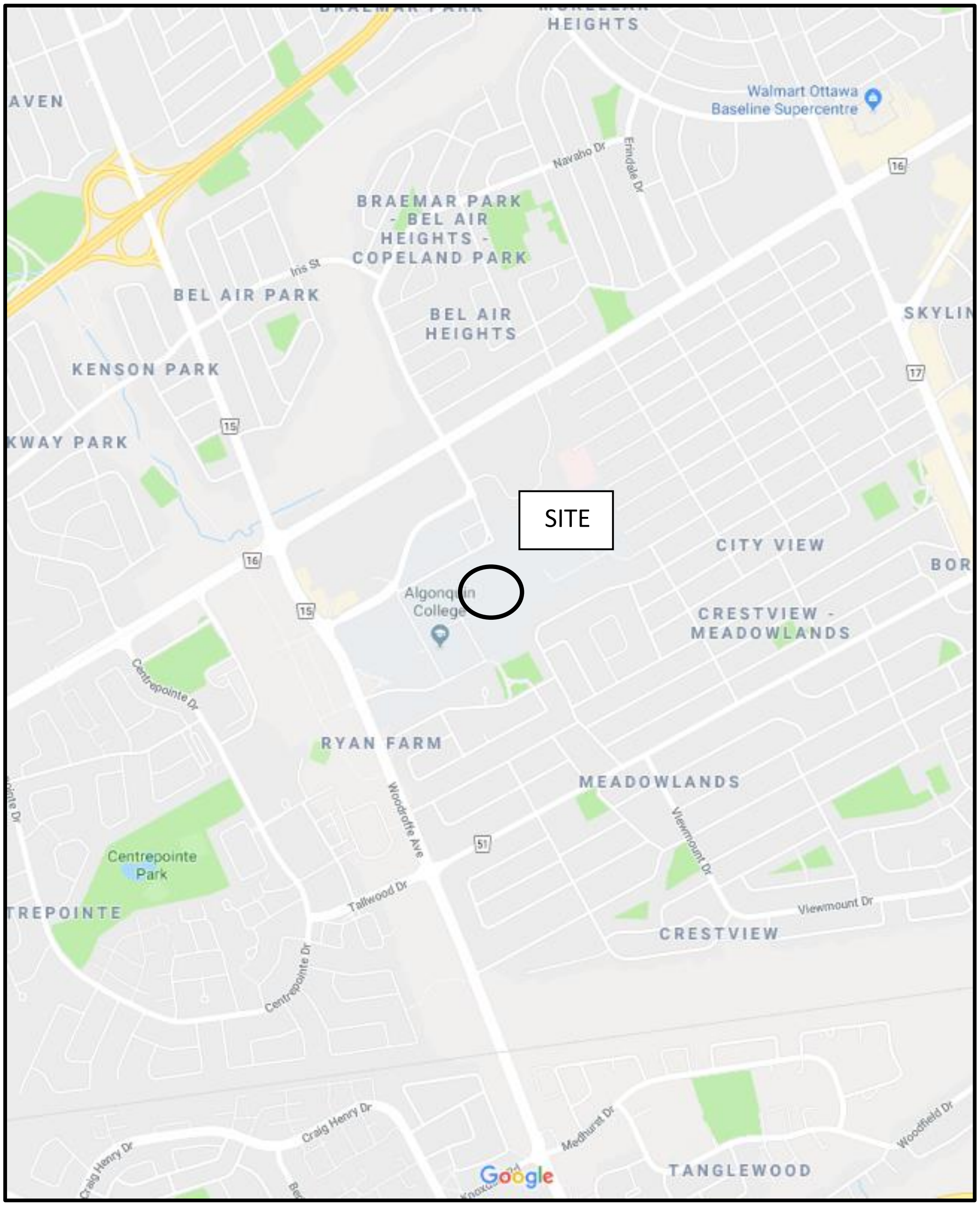


Figure 1

Key Plan

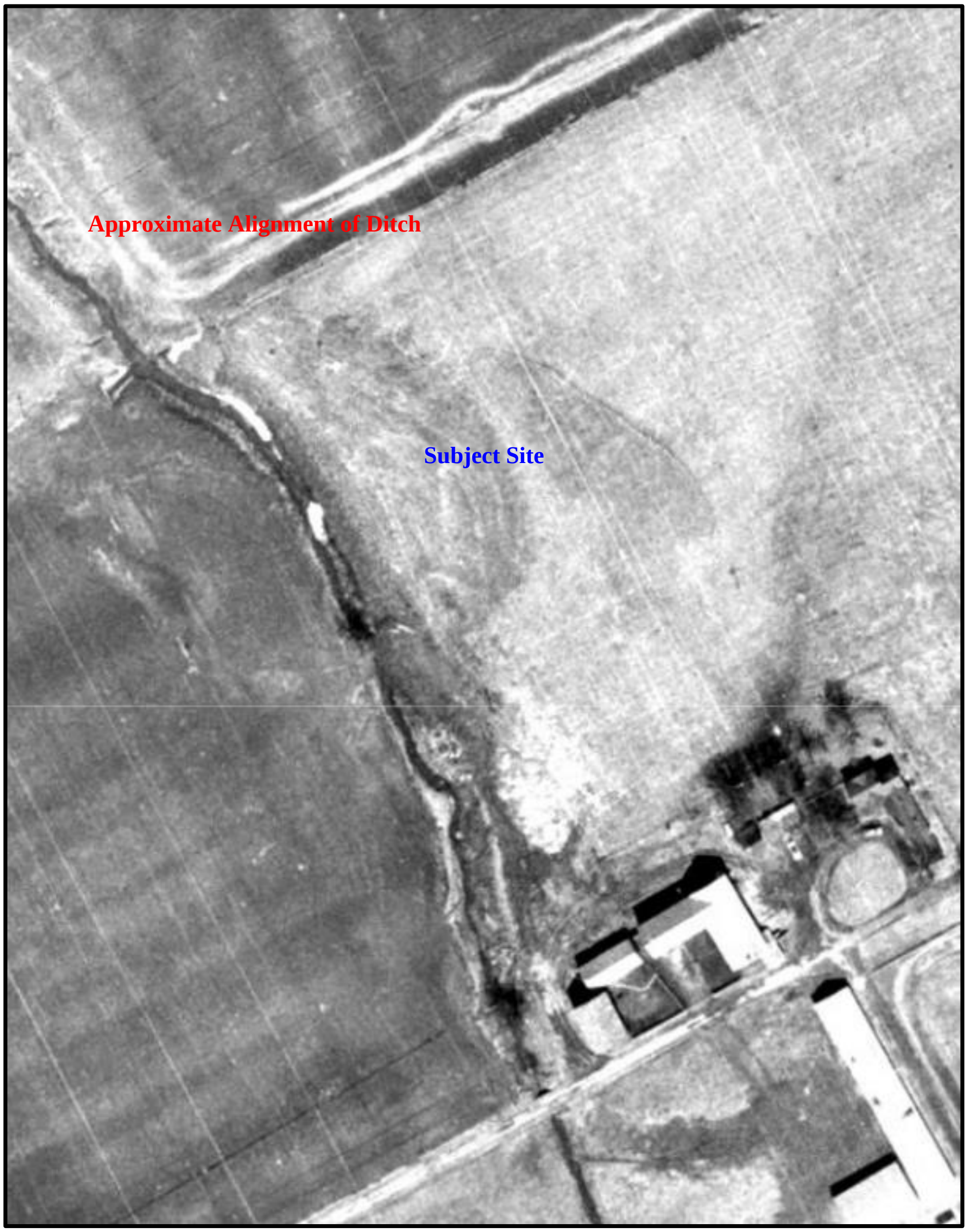


Figure 2

Aerial Photograph - 1965

Figure 3

Aerial Photograph - 1991

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Figure 4

Aerial Photograph - 2017

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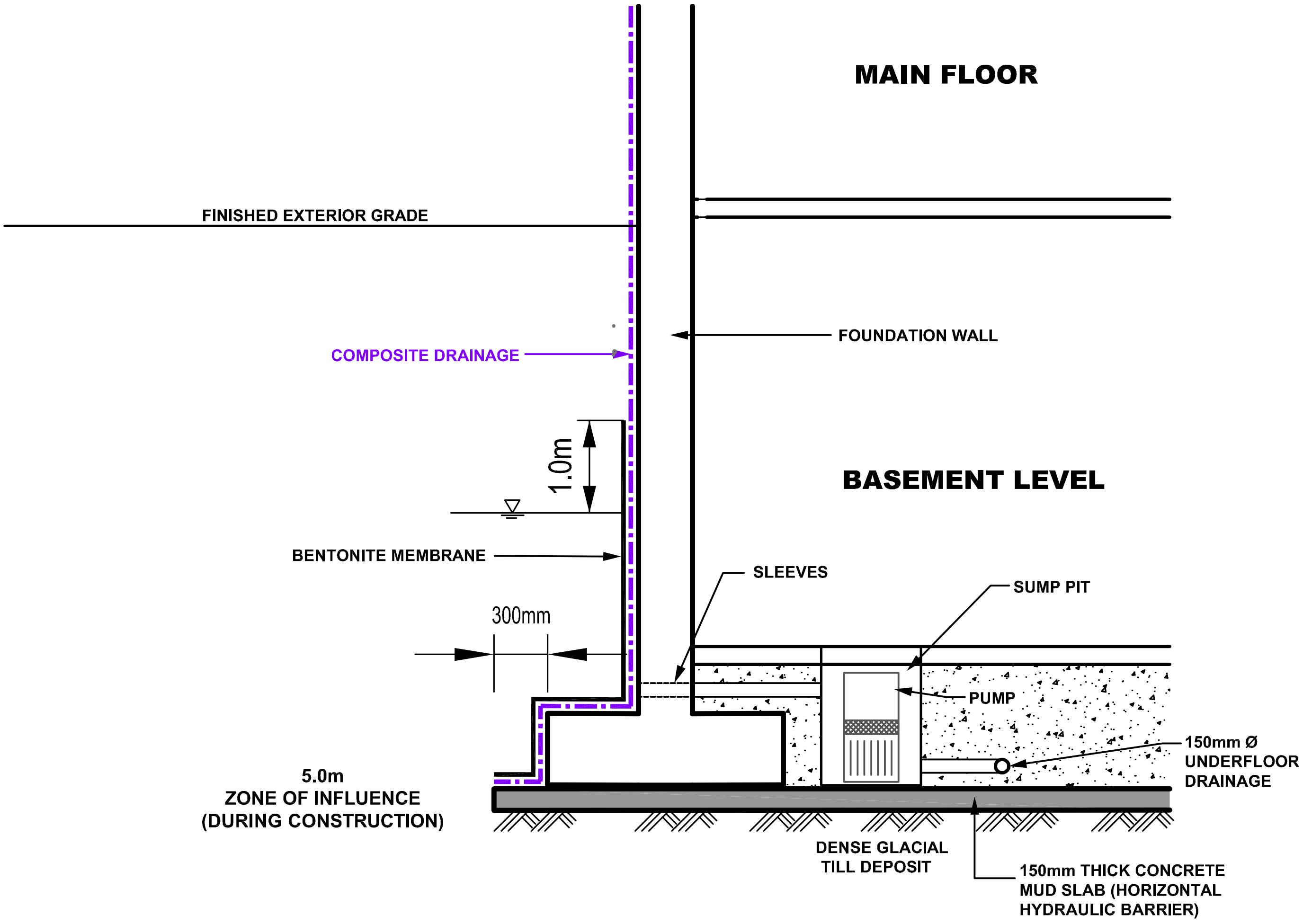

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ALGONQUIN COLLEGE

PROP. ATHLETIC RECREATION COMPLEX
WOODROFFE CAMPUS, WOODROOFE AVE.
OTTAWA, ONTARIO

Title:
WATER SUPPRESSION SYSTEM

Date: **05/2019**

Report No.: **PG4624**

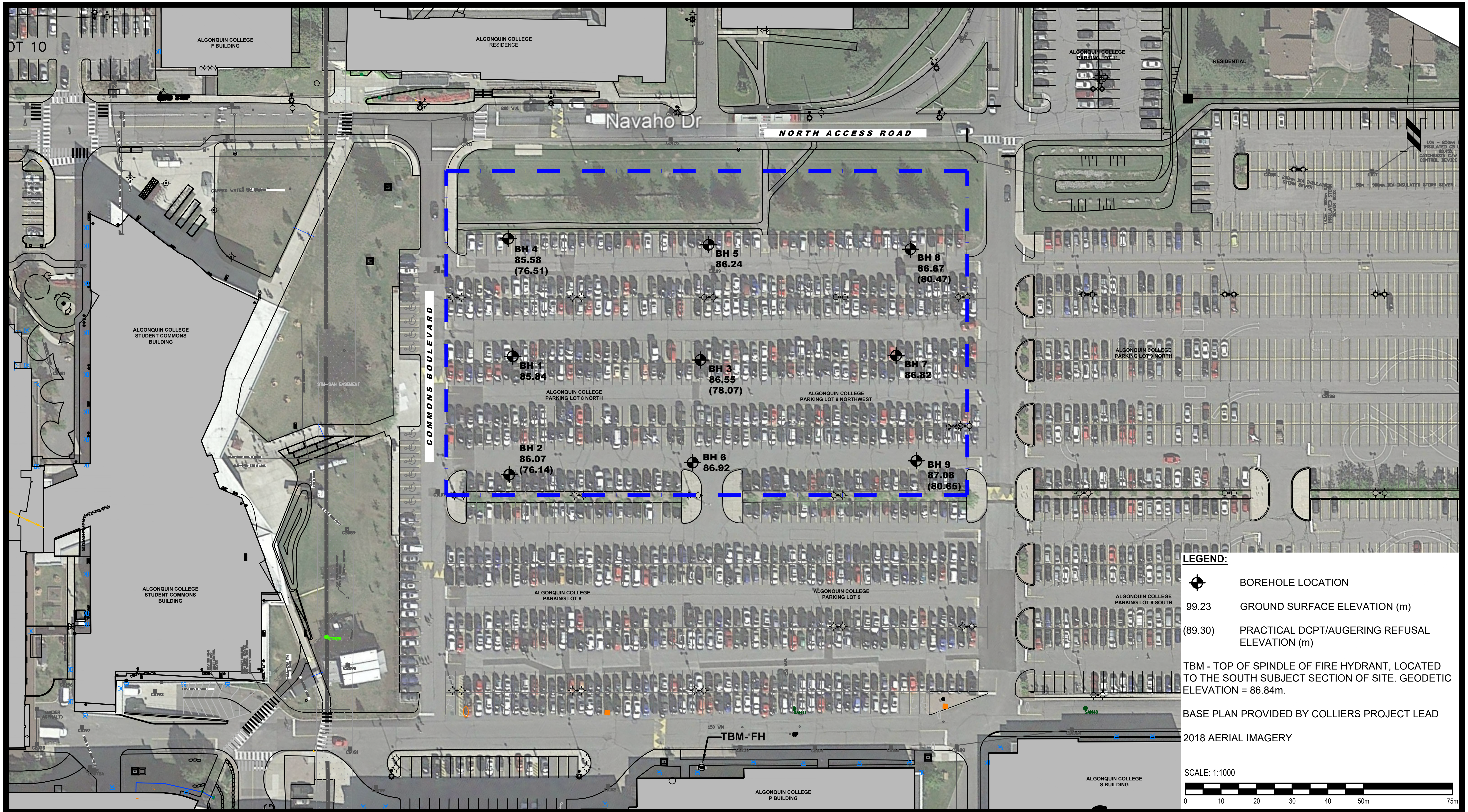
Scale: **N.T.S.**

Drawing No.: **FIGURE 5**

Drawn by: **RCG**

Checked by: **CDS**

p:\autocad drawings\geotechnical\pg46xx\pg4624\pg4624 water suppression system.dwg



LEGEND:

⌖	BOREHOLE LOCATION
99.23	GROUND SURFACE ELEVATION (m)
(89.30)	PRACTICAL DCPT/AUGERING REFUSAL ELEVATION (m)

TBM - TOP OF SPINDLE OF FIRE HYDRANT, LOCATED TO THE SOUTH SUBJECT SECTION OF SITE. GEODETIC ELEVATION = 86.84m.

BASE PLAN PROVIDED BY COLLIERS PROJECT LEAD

2018 AERIAL IMAGERY

SCALE: 1:1000

0 10 20 30 40 50m 75m

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
1	GROUND SURFACE ELEVATIONS CONVERTED TO GEODETIC	05/31/2019	FA

COLLIERS PROJECT LEADERS
GEOTECHNICAL INVESTIGATION - PROPOSED ATHLETIC RECREATION COMPLEX (ARC)
ALGONQUIN COLLEGE - WOODROFFE AVENUE
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1000	Date: 09/2018
Drawn by: RCG	Report No.: PG4624-1
Checked by: RG	Dwg. No.: **PG4624-1**
Approved by: DJG	Revision No.: 1

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