Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

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Geotechnical Investigation

Proposed Multi-Storey Office Building
800 Palladium Drive
Ottawa, Ontario

Prepared For

Cominar

May 1, 2019

Report PG4802-1 Revision 2

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

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Cominar to complete a geotechnical report based on existing borehole information, for the proposed multi-storey office building to be located at 800 Palladium Drive, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

❏ Review subsoil and groundwater information at this site.

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

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

2.0 Proposed Project

The proposed project will consist of a multi-storey (5 floors) office building of slab on grade construction. It is expect that the project will include associated access roads, landscape areas and at grade parking 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 April 10, 2012. At that time, three (3) boreholes were completed across the subject site. The borehole locations were selected by Paterson taking in consideration the site features at the time of investigation. The locations of the boreholes are shown on Drawing PG4802-1 - 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 Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

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

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

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

Overburden thickness was evaluated during the course of the investigation by dynamic cone penetration testing (DCPT) at BH 1. 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 boreholes 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 in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

3.2 Field Survey

The borehole locations were selected in the field by Paterson personnel in a manner to provide general coverage of the proposed building, taking into consideration site features and underground utilities. The ground surface elevations at the test hole locations were referenced to a temporary benchmark (TBM), consisting of the top spindle of the fire hydrant located on the north side of Palladium Drive, just east from Cyclon Taylor Boulevard. An arbitrary elevation of 100 m was assigned to the TBM. The location of the boreholes, the TBM and the ground surface elevations at each test hole location are presented on Drawing PG4802-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 OBSERVATIONS

4.1 Surface Conditions

The site is relatively flat, gravel covered and at grade with surrounding streets and properties. A maximum difference in ground surface elevation of 0.4 m was measured among the borehole locations. The site is currently being used as a parking lot.

4.2 Subsurface Profile

Generally, the subsurface profile consists of granular fill followed by silty sand with clay which in turn is underlain by stiff to very stiff brown silty clay followed by firm silty clay. Based on the DCPT glacial till was inferred at a 13 m depth at BH 1. Practical refusal to DCPT was encountered at a 17 m depth at BH 1. Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Based on available geological mapping, interbedded limestone and shale bedrock of the Verulam Formation is present in this area with an overburden thickness ranging between 15 to 25 m. It is our understanding that bedrock was previously encountered at a depth of approximately 24 m during the piling operations for nearby buildings.

4.3 Groundwater

Groundwater levels were measured in the piezometers standpipes on April 16, 2012. It should be noted that groundwater level readings in a low permeability soil can result in higher than normal groundwater levels due to surface water becoming trapped in a backfilled borehole. Long-term groundwater levels can also be estimated based on the observed color, consistency and moisture content of the recovered soil samples. Based on these observations, it is estimated that groundwater can be expected between 2 to 3 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. Conventional footing foundations can be considered provided the bearing resistance values are compliant with the anticipated building loads. Where design loads exceed the given bearing resistance, consideration may be given to a foundation founded on end bearing piles or a raft foundation.

Due to the presence of the deep silty clay deposit, grade raise restrictions are required to reduce the risk of detrimental long-term total and differential settlements.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any building and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the proposed building, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It 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 area 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.

Existing engineered fill is suitable for re-use as backfill material against foundation walls in combination a perimeter drainage system and under pavement structures. Site excavated material for re-use shall be approved by Paterson at the time of construction and stored on site in a way to avoid water infiltration and freezing.

5.3 Foundation Design

Spread Footing Foundation

Consideration may be given to placing the footings for a 5 storey building design at a higher level taking advantage of the stiff silty clay material. Shallow pad footings, up to 6 m wide, founded on an undisturbed stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **125 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **200 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS.

For design purposes, a permissible grade raise of 1.0 m above existing ground surface is recommended for the proposed building.

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.

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

Raft Foundation

Consideration can be given to a raft foundation if the building loads are acceptable. The following parameters may be used for raft design.

For design purposes, the factored bearing resistance at ULS can be taken as **100 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. A bearing resistance value at SLS (contact pressure) of **70 kPa** can be used. 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 modulus of subgrade reaction was calculated to be **2.8 MPa/m** for a contact pressure of **70 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

The proposed building can be designed using the above parameters and a total and differential settlement of 25 and 20 mm, respectively.

Deep Foundation

For support of the proposed multi-storey building consideration could be given to using concrete filled steel pipe piles driven to refusal on the bedrock surface.

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 1. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of two (2) to four (4) piles would be recommended. As a minimum, the pipe piles should be equipped with a base plate having a thickness of at least 20 mm to minimize damage to the pile tip during driving. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 1 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	940	1130	10	29
245	11	1175	1410	10	35
245	13	1375	1650	10	42

5.4 Design for Earthquakes

A seismic site response **Class D** should be used for design of the proposed building at the subject site according to the OBC 2012. The soils underlying the site are not susceptible to liquefaction.

5.5 Rock Anchor Design

In the event that rock anchors are required for lateral shear wave for seismic considerations, the following geotechnical design parameters are provided. The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada) or Williams Form Engineering, have qualified personnel on staff to recommend appropriate rock anchor size and materials.

It should be further noted that centre to centre spacing between bond lengths be at least four (4) times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service. To resist seismic uplift pressures, a passive rock anchor system can be used. It should be noted that a post-tensioned anchor will take the uplift load with much less deflection than a passive anchor.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

Grout to Rock Bond

Generally, the unconfined compressive strength of limestone/dolomite ranges between about 60 and 90 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.2 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing subsoils information, a **Rock Mass Rating (RMR) of 50** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.128 and 0.00009**, respectively. For design purposes, we assumed that all rock anchors will be placed at least 1.2 m apart to reduce group anchor effects.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2.

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.2 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Fair quality Dolostone Hoek and Brown parameters	50 m=0.128 and s=0.00009
Unconfined compressive strength - Dolomite bedrock	60 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

Based on our calculations, **where rock anchor interaction is minimized (ie.- >1.2 m apart),** a **6 m total anchor length** and a **3 m fixed anchor length** is adequate to resist a **890 kN force**. As well, a **9 m total anchor length** and a **4 m fixed anchor length** is adequate to resist a **1,780 kN load**, where anchor interaction is not encountered.

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

It is should be noted that due to the intended use of the rock anchors and nature of the passive rock anchor design, proof testing is not required provided that the grout installation is adequately completed to the satisfaction of the geotechnical consultant. It is recommended that compressive strength testing be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

5.6 Slab on Grade Construction

With the removal of all topsoil and fill, containing deleterious materials 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 the floor slab. The upper 150 mm of sub-slab fill should consist of an OPSS Granular A material for slab on grade construction. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

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

It should be noted that the subgrade material will consist of the sensitive silty clay which is susceptible to disturbance in the presence of water and/or under the traffic of personnel and vehicles. It is recommended that the bottom of the excavation be protected using a granular pad or lean concrete as soon as possible following the completion of the excavation. Similarly, a compacted granular working pad should be provided for the piling operations to permit the movement of the piling rig and other construction traffic.

5.7 Pavement Structure

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

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

Table 4 - Recommended Pavement Structure Access Lanes 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
400	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

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

Pavement Structure Drainage

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

Consideration should be given to installing subdrains during the pavement construction. These drains should be installed on both sides of the pavement with their inverts approximately 300 mm below the subgrade level. The drains should be connected to a positive outlet. The subgrade surface should be crowned to promote water flow to the drainage lines. The subdrains will help drain the pavement structure, especially in early Spring when the subgrade is saturated and weaker and, therefore, more susceptible to permanent deformation.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structure to ensure that frost heave sensitive sidewalks adjacent to the building have adequate drainage for the sub-soils. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the sub-grade level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. 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 a composite drainage system, such as Miradrain G100N, is provided.

6.2 Protection of Footings, Pile Caps and Grade Beams Against Frost Action

Footings, pile caps and grade beams of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided in this regard.

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

6.3 Excavation Side Slopes

The side slopes of 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. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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 kept away from the excavation sides.

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

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

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 & Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material’s SPMDD.

It should generally be possible to re-use the moist, not wet, site excavated brown silty clay or silty sand above the cover material if the excavation and filling operations are carried out in dry weather conditions. It may not be practical to re-use wet silty clay as compacting this material without an extensive drying period may be impractical.

Where hard surfaces are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below the finished grade) should match the soils exposed at the trench walls to reduce 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 clay seals should be as per Standard Drawing S8 of the Department of Public Works & Services - Infrastructure Services Branch of the City of Ottawa. The seals should be at least 1.5 m long (in the trench direction), as compared to the 1 m minimum in the detail, 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 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

Due to the relatively impervious nature of the silty clay materials, 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 shallow excavations.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 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.

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.

6.6 Winter Construction

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

The subsoil conditions at this site mostly consist of frost susceptible materials. In 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.

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

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

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

- ❏ Observation of all bearing surfaces prior to the placement of concrete.
- ❏ Sampling and testing of the concrete and fill materials used.
- ❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❏ Observation of all subgrades prior to backfilling.
- ❏ Field density tests to determine the level of compaction achieved.
- ❏ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

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 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 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 Cominar or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
May 1,2019
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Joey R Villeneuve, M.A.Sc, EIT

David J. Gilbert, P.Eng.

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Building - 800 Palladium Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northeast corner of Palladium Drive and Cyclone Taylor Blvd. Elevation = 96.85m, as provided by Novatech Consulting Engineers & Planners.

REMARKS

BORINGS BY CME 55 Power Auger

DATE April 10, 2012

FILE NO. PG4802

HOLE NO. BH 1

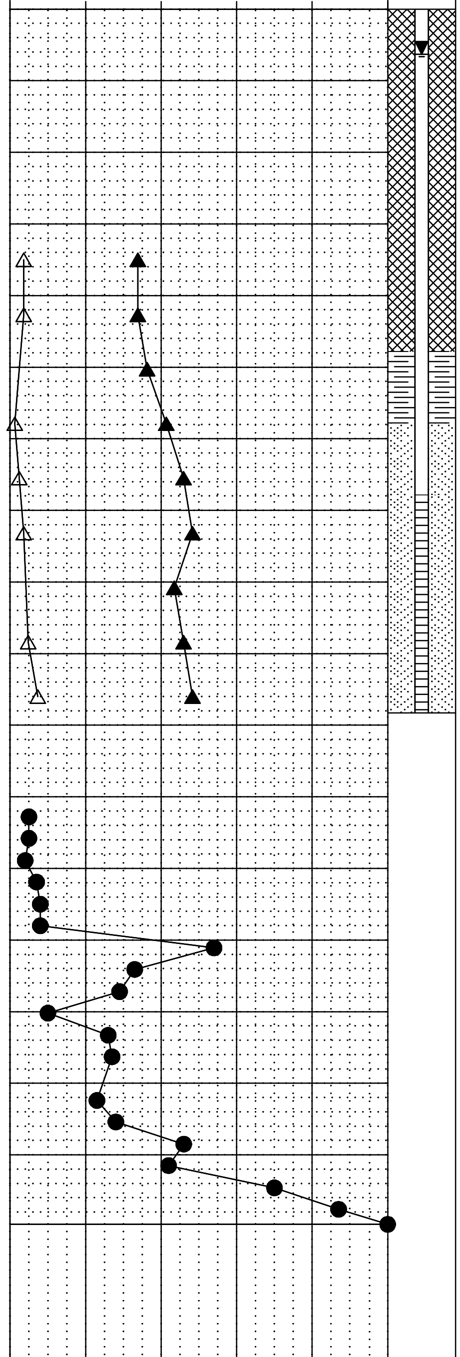
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	94.99
FILL: Crushed stone — 0.60							
Grey **SILTY SAND** with clay — 1.45		SS	1	42	7	1	93.99
Stiff, brown **SILTY CLAY** — 2.21		SS	2	100	2	2	92.99
Firm, grey **SILTY CLAY** — 9.83		SS	3	100	1	3	91.99
						4	90.99
						5	89.99
						6	88.99
						7	87.99
						8	86.99
						9	85.99
Dynamic Cone Penetration Test commenced at 9.83m depth. — 16.97						10	84.99
						11	83.99
						12	82.99
						13	81.99
						14	80.99
						15	79.99
						16	78.99
End of Borehole Practical refusal to DCPT at 16.97m depth (GWL @ 0.63m-April 16, 2012)							

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

○ Water Content %

20 40 60 80

Piezometer Construction



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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Building - 800 Palladium Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northeast corner of Palladium Drive and Cyclone Taylor Blvd. Elevation = 96.85m, as provided by Novatech

REMARKS Consulting Engineers & Planners.

FILE NO. PG4802

HOLE NO. BH 2

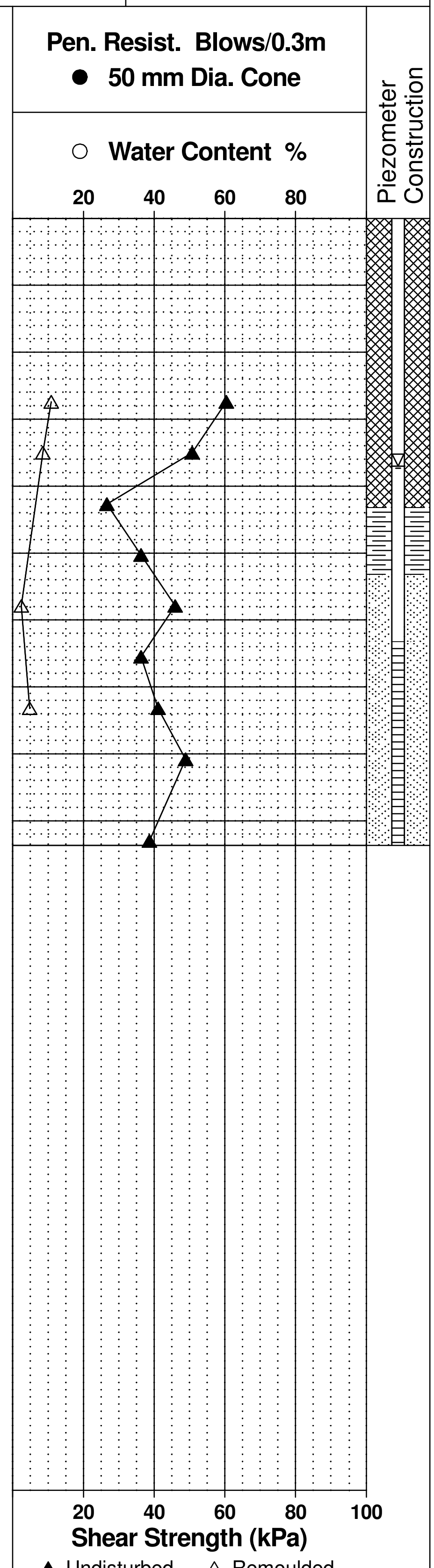
BORINGS BY CME 55 Power Auger **DATE** April 10, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	95.47
FILL: Crushed stone — 0.60							
Stiff, brown **SILTY CLAY** with sand — 1.45		SS	1	0	7	1	94.47
Stiff, brown **SILTY CLAY** - firm and grey by 3.7m depth — 9.37		SS	2	42	3	2	93.47
						3	92.47
						4	91.47
						5	90.47
						6	89.47
						7	88.47
						8	87.47
						9	86.47

End of Borehole

(GWL @ 3.6m depth based on field observations)

(Piezometer damaged - GWL reading not obtained)



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Commercial Building - 800 Palladium Drive
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the northeast corner of Palladium Drive and Cyclone Taylor Blvd. Elevation = 96.85m, as provided by Novatech

REMARKS Consulting Engineers & Planners.

FILE NO. PG4802

HOLE NO. BH 3

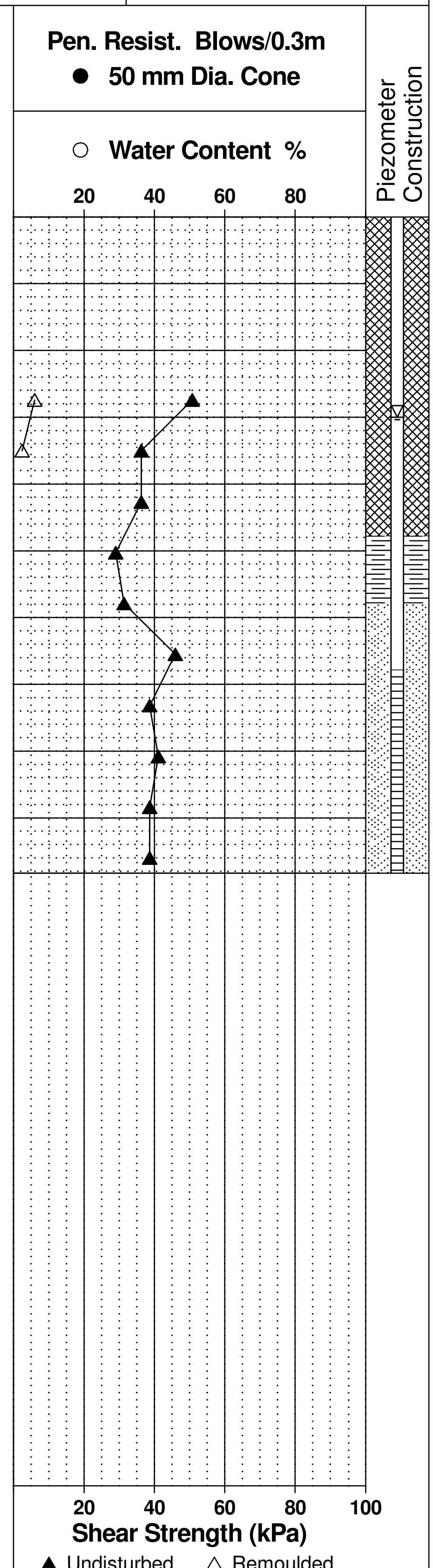
BORINGS BY CME 55 Power Auger **DATE** April 10, 2012

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Crushed stone	0.60				
Compact, brown **SILTY SAND,** trace clay	1.45	SS	1	42	10
Stiff, brown **SILTY CLAY**		SS	2	100	2
- firm and grey by 3.0m depth	9.83				
End of Borehole					

DEPTH (m)	ELEV. (m)
0	95.03
1	94.03
2	93.03
3	92.03
4	91.03
5	90.03
6	89.03
7	88.03
8	87.03
9	86.03

(GWL @ 3.0m depth based on field observations)

(Piezometer damaged - GWL reading not obtained)



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

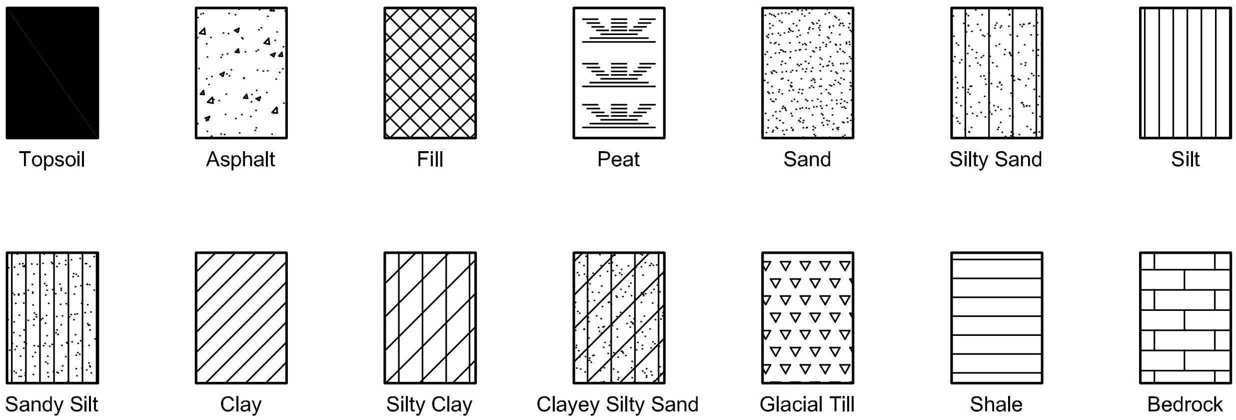
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

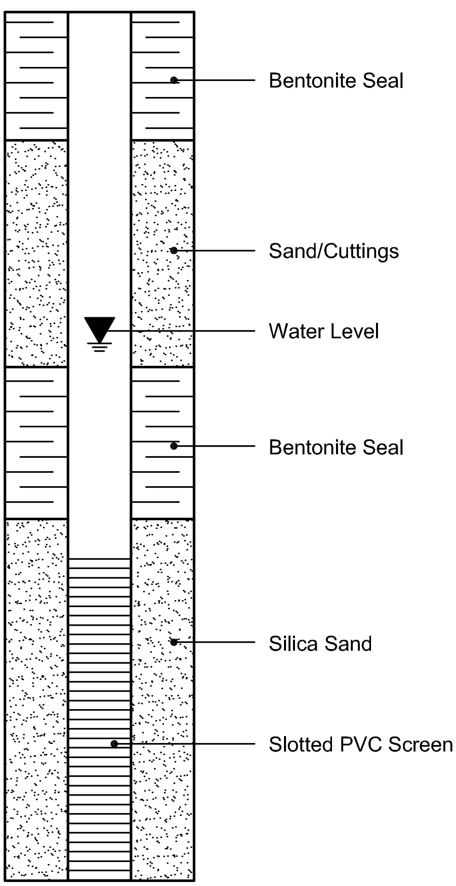
SYMBOLS AND TERMS (continued)

STRATA PLOT

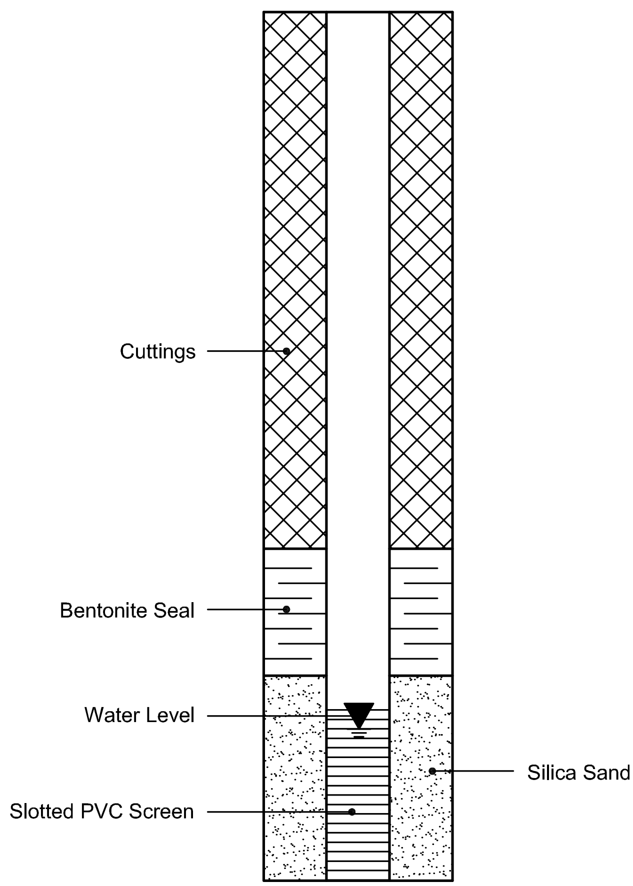


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4802-1 - TEST HOLE LOCATION PLAN

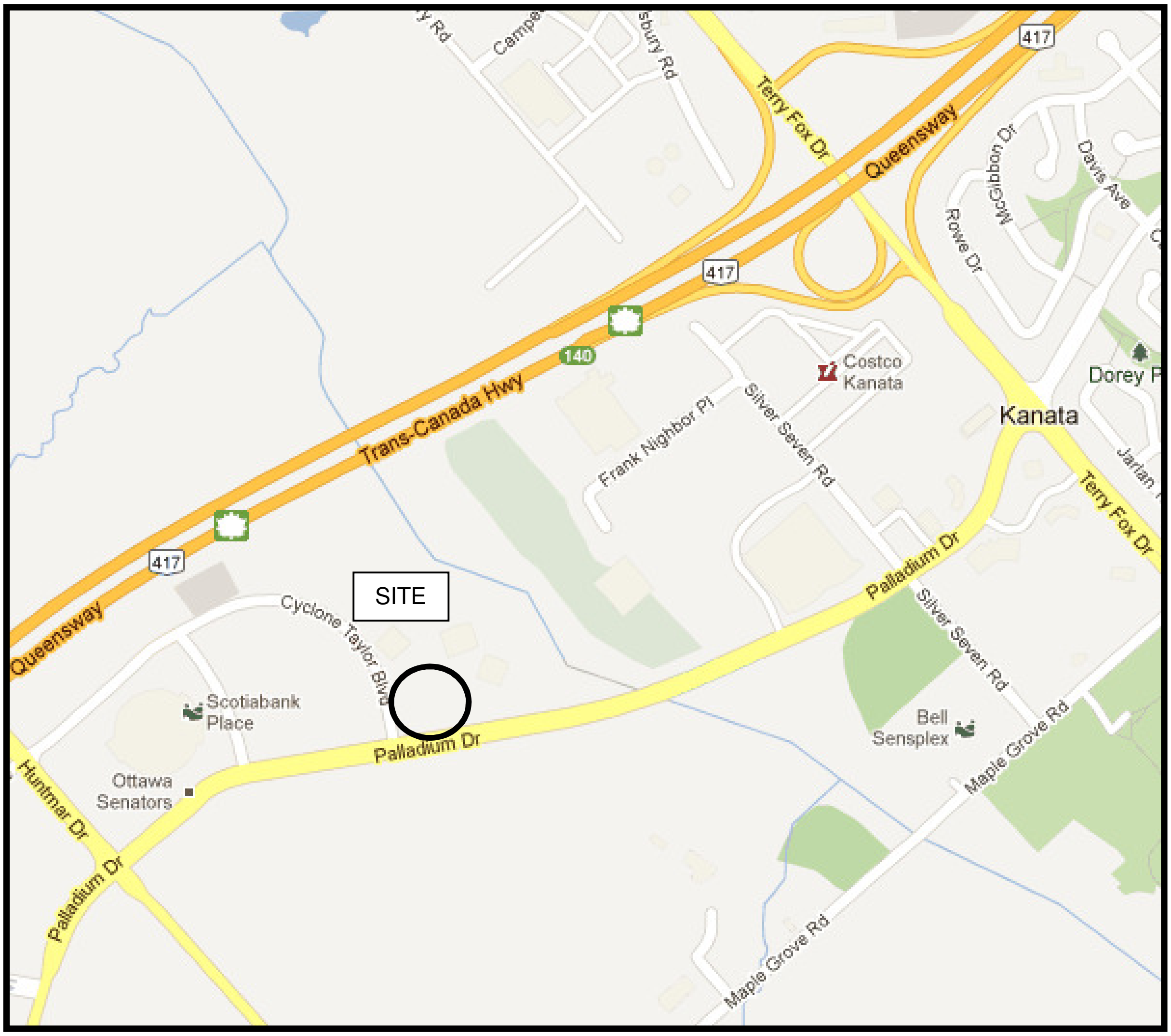
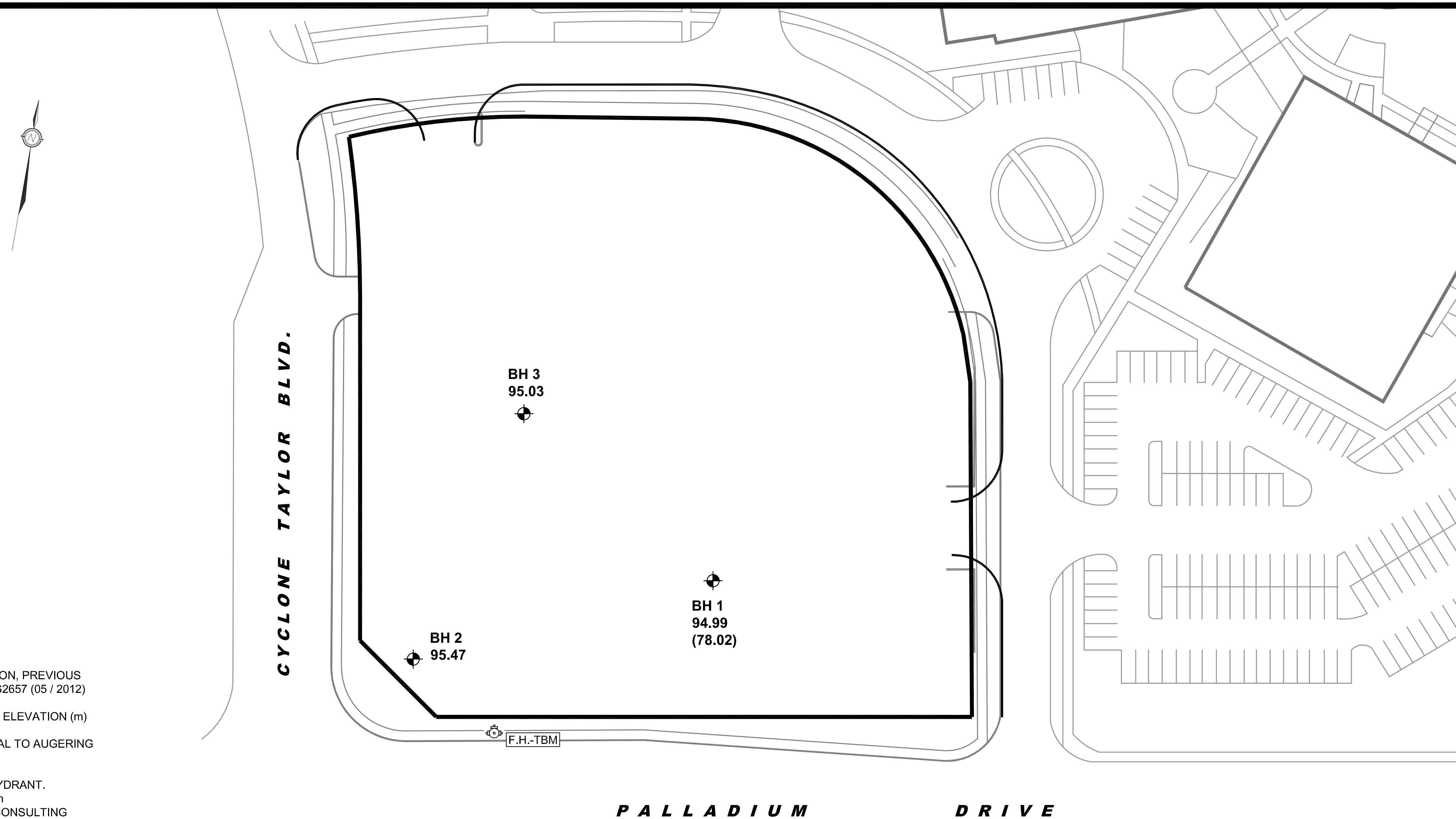


FIGURE 1

KEY PLAN

patersongroup



LEGEND:

- BOREHOLE LOCATION, PREVIOUS INVESTIGATION, PG2657 (05 / 2012)
- 94.99 GROUND SURFACE ELEVATION (m)
- (78.2) PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

TBM - TOP SPINDLE OF FIRE HYDRANT.
GEODETIC ELEVATION = 96.85m
AS PROVIDED BY NOVATECH CONSULTING ENGINEERS AND PLANNERS.

BASE PLAN PROVIDED BY NOVATECH CONSULTING ENGINEERS AND PLANNERS.

SCALE: 1:750

0 10 20 30 50m

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

COMINAR REIT
GEOTECHNICAL INVESTIGATION
PROP. COMMERCIAL BUILDING - 800 PALLADIUM DRIVE
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 01/2019
Drawn by: RCG	Report No.: PG4802-1
Checked by: JV	Dwg. No.: PG4802-1
Approved by: DJG	Revision No.: 0

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