Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

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patersongroup

Geotechnical Investigation

Proposed Residential Development
Wateridge - Block 29
Wanaki Road - Ottawa

Prepared For

Colonnade Bridgeport

June 21, 2019

Report: PG4965-1

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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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Colonnade Bridgeport to conduct a geotechnical investigation for the proposed residential development to be located at the northwest corner of the intersection of Wanaki Road and Provender Avenue, in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the investigation was to:

- determine the subsurface soil and groundwater conditions by means of test holes.
- provide geotechnical recommendations for the foundation design for the proposed buildings and pavement structure design of the proposed development including construction considerations which may affect the design.

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

It is understood that the proposed development will consist of a residential buildings with one basement level. Local roadways and car only parking areas are also anticipated for the proposed development. It is further anticipated that the site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current investigation was carried out on May 6, 2019. At that time, 5 test pits were excavated to a maximum depth of 4.4 m below existing ground surface. The test hole locations were placed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations for the current investigation are presented on Drawing PG4965-1 - Test Hole Location Plan included in Appendix 2.

The test pits were excavated using a hydraulic shovel and backfilled with the excavated soil upon completion. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division. The testing procedure consisted of augering to the required depths and at the selected locations sampling the overburden.

Sampling and In Situ Testing

Grab samples were collected from the test pit at selected intervals. All samples were transported to our laboratory. The depths at which the grab samples were recovered from the test pit are depicted as G on the Soil Profile and Test Data sheet in Appendix 1.

Undrained shear strength testing in test pits was completed using a handheld, portable vane apparatus (field inspection vane tester Roctest Model H-60).

The subsurface conditions observed in the test pit were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

3.2 Field Survey

The test pit locations were determined by Paterson personnel taking into consideration site features. The location and ground surface elevation at each test pit location were referenced to a temporary benchmark (TBM) consisting of the top spindle of a fire hydrant located along Wanaki Road. An assumed elevation of 100 m was assigned to the TBM. The test hole locations and ground surface elevations at the test hole locations are presented on Drawing PG4965-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The ground surface across the subject site is currently gravel covered and generally lower than the surrounding roadways by approximately 2 m. Wick drains were observed to be installed along the east portion of the subject site within the granular fill layer. The ground surface across the subject site is relatively flat within the centre and slopes up towards the edges.

The site is bordered by Wanaki Road to the east, Provender Avenue to the south, Burma Road to the west and a gravel access road followed by a Stormwater Management pond to the north.

4.2 Subsurface Profile

Generally, the soil profile encountered at the test hole locations consists of topsoil and/or fill consisting of crushed stone followed by hard to very stiff silty clay crust. A stiff to firm grey silty clay deposit was encountered below the above noted layers. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Based on available geological mapping, the bedrock consists mainly of interbedded limestone, dolomite and shale from the Gull River, Bobcaygeon or Rockcliffe formations. The overburden drift thickness ranges between 1 and 15 m depth.

4.3 Groundwater

Based on observations during excavation of the test pits, moisture levels and colour of the recovered samples, the long-term groundwater level is expected between 2 and 3 m below existing grade. It should be noted that groundwater levels are subject to seasonal fluctuations, therefore the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is satisfactory for the proposed residential development. It is expected that the proposed building will be founded over conventional shallow footings placed on an undisturbed, hard to very stiff silty clay or approved fill bearing surface.

Due to the presence of the a silty clay layer, the proposed development will be subjected to grade raise restrictions. A permissible grade raise of **2 m** is recommended over original ground surface for the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

For building footprints where the existing fill, free of significant amounts of organics and over-sized boulders/concrete pieces, is encountered, the following procedure is recommended:

❏ The existing fill should be sub-excavated to a minimum 500 mm depth below the underside of the floor slab. The excavated fill material, free of significant amounts of organics and over-sized boulders/concrete, can be re-compacted in maximum 200 mm thick lifts using a sheepsfoot roller making several passes. The re-compaction effort should be carried out under dry conditions and above freezing temperatures and be supervised by the geotechnical consultant.

❏ If the existing fill is encountered below underside of footing, it is recommended to remove the existing fill material extending at least 500 mm below the underside of footing. The approved fill subgrade, free of significant amounts of organics and over-sized boulders/concrete, should be proof-rolled by a sheeps foot roller making several passes. Any poor performing areas should be removed. The sub-excavated area should be reinstated with engineered fill, such as OPSS Granular A or Granular B Type II, compacted in maximum 300 mm thick loose lifts to 98% of the SPMDD.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. Granular material should be tested and approved prior to delivery to the site. The fill should be placed in loose lifts of 300 mm thick or less and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of the Standard Proctor Maximum Dry Density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath parking areas where settlement of the ground surface is of minor concern. In landscaped areas, 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 the SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values

Pad footings up to 5 m wide, and strip footings, up to 3 m wide, placed on an undisturbed , hard to very stiff silty clay bearing surface should be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

Footings placed on an approved fill bearing surface can be designed using a bearing resistance value at SLS of **100 kPa** and a factored bearing resistance value at ULS of **175 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

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, prior to the placement of concrete for footings.

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the in-situ bearing medium soils above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Permissible Grade Raise Recommendations

Based on the existing test hole information, a **permissible grade raise restriction of 2 m** is recommended for the subject site. A post-development groundwater lowering of 0.5 m was considered in the permissible grade raise restriction calculations.

5.4 Design for Earthquakes

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

5.5 Basement Slab

With the removal of all topsoil and deleterious fill, containing organic matter, within the footprints of the proposed buildings, the native soil surface or approved fill will be considered an acceptable subgrade on which to commence backfilling for floor slab construction. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consist of 19 mm clear crushed stone.

Where existing fill is encountered beneath the basement slab, these areas should be reviewed as per our recommendations presented in Subsection 4.2.

5.6 Pavement Structure

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

Table 1 - 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 2 - Recommended Pavement Structure - Local Residential Roadways

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
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 driveways and local roadways.

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 material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on 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 load carrying capacity.

Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

A perimeter foundation drainage system is recommended for proposed structures. The system should consist of a 100 to 150 mm diameter, geotextile-wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing 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. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as Miradrain G100N or Delta Drain 6000) connected to a drainage system is provided.

6.2 Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum 1.5 m thick soil cover (or equivalent) should be provided in this regard.

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other exterior unheated footings.

6.3 Excavation Side Slopes

The excavations for the proposed development will be mostly through a sensitive grey silty clay. Where excavation is above the groundwater level to a depth of approximately 3 m, the excavation side slopes should be stable in the short term at 1H:1V. Flatter slopes could be required for deeper excavations or for excavation below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring should be used. The subsoil at this site is considered to be mainly a Type 2 or 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.

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.

It is expected that deep service trenches in excess of 3 m will be completed using a temporary shoring system designed by a structural engineer, such as stacked trench boxes in conjunction with steel plates. The trench boxes should be installed to ensure that the excavation sidewalls are tight to the outside of the trench boxes and that the steel plates are extended below the base of the excavation to prevent basal heave (if required).

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.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the City of Ottawa. These recommendations are for standard, open cut excavation placed services.

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located within the firm grey silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extent at least to the spring line of the pipe.

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

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

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long (in the trench direction) and should extend from trench wall to trench wall. The seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the 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. Periodic inspection of the clay seal placement work should be completed by Paterson personnel during servicing installation work.

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 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

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

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

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

6.7 Corrosion Potential and Sulphate

One (1) sample was submitted for testing. The analytical test results of the soil sample indicate that the sulphate content is less than 0.01%. These results along with the chloride and pH value are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The results of the resistivity indicate the presence of a moderate to very aggressive environment for exposed ferrous metals at this site, which is typical of silty clay samples submitted for the subject area. It is anticipated that standard measures for corrosion protection are sufficient for services placed within the silty clay deposit.

6.8 Landscaping Considerations

Tree Planting Restrictions

Based on the undrained shear strength values measured in the silty clay deposit within all the test pits, the silty clay within the subject site is not considered sensitive marine clay. Therefore, the proposed residential dwellings are located in a low to moderate sensitivity area with respect to tree planting over a silty clay deposit. It is recommended that trees placed within 4.5 m of the foundation wall consist of low water demanding trees with shallow roots systems that extend less than 1.5 m below ground surface. Trees placed greater than 4.5 m from the foundation wall may consist of typical street trees, which are typically moderate water demand species with roots extending to a maximum 2 m depth.

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e. Manitoba Maples) and, as such, they should not be considered in the landscaping design.

7.0 Recommendations

It is recommended that the following be completed once the master plan and site development are determined:

- ❑ Review detailed grading plan(s) from a geotechnical perspective.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ 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 placing backfilling materials.
- ❑ Observation of clay seal placement at specified locations.
- ❑ Field density tests to ensure that the specified level of compaction has been 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 Paterson's recommendations could be issued upon request, following the completion of a satisfactory material testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with Paterson's present understanding of the project. Paterson requests permission to review the grading plan once available. Paterson's recommendations should be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and the test hole log are furnished as a matter of general information only. Test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests to be notified immediately in order to permit reassessment of the 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 Colonnade Bridgeport or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
June 21, 2019
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Faisal I. Abou-Seido, P.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TEST RESULTS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - Wateridge Block 29
Burma Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant along Wanaki Road. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY Hydraulic Shovel

DATE 2019 May 6

FILE NO. PG4965

HOLE NO. TP 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.18
FILL: Crushed stone							
0.55		G	1				
						1	97.18
Very stiff to hard, grey-brown **SILTY CLAY**		G	2				
		G	3				
						2	96.18
		G	4				
						3	95.18
3.10		G	5				
End of Test Pit							
(Groundwater infiltration at 1.9m depth)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

Shear strength values plotted (top to bottom):

122
116
116
108
140
196
236
260
232
224
228

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - Wateridge Block 29
Burma Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant along Wanaki Road. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

BORINGS BY Hydraulic Shovel

DATE 2019 May 6

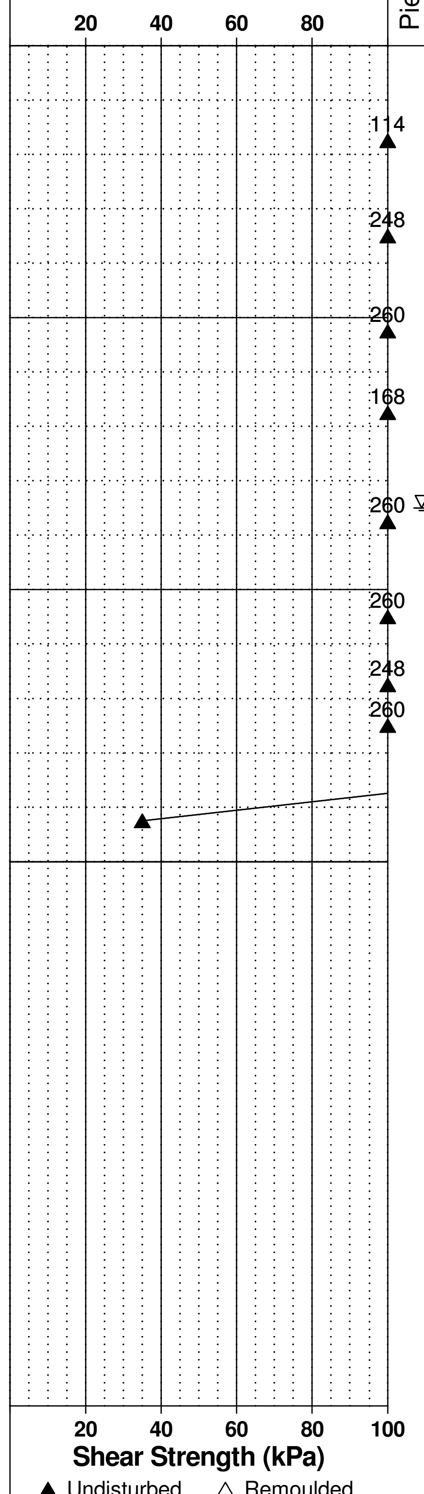
FILE NO. PG4965

HOLE NO. TP 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.94
TOPSOIL 0.25		G	1				
Very stiff to hard, grey **SILTY CLAY**		G	2				
		G	3			1	96.94
		G	4				
						2	95.94
		G	5				
- firm and grey by 2.7m depth 3.00		G	6			3	94.94
End of Test Pit							
(Groundwater infiltration at 1.7m depth)							

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

○ Water Content %



Piezometer Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - Wateridge Block 29
Burma Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant along Wanaki Road. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

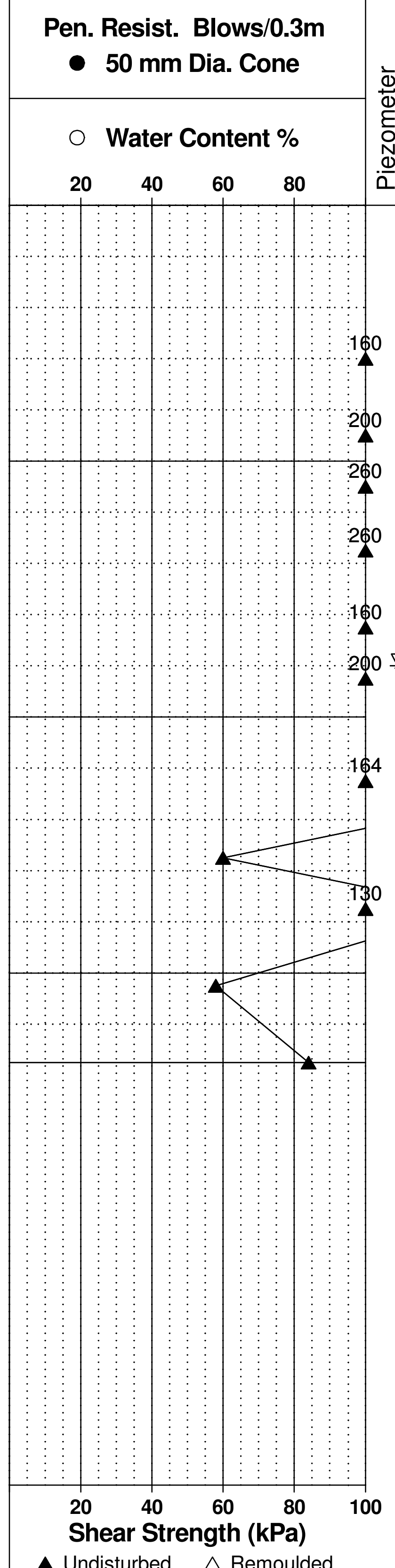
BORINGS BY Hydraulic Shovel

DATE 2019 May 6

FILE NO. PG4965

HOLE NO. TP 3

SOIL DESCRIPTION	Depth to base (m)	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							0	97.84
FILL: Sandy gravel with silty clay and stone dust	0.15		G	1				
TOPSOIL	0.45		G	2				
Hard to very stiff, grey-brown **SILTY CLAY**			G	3				
			G	4			1	96.84
			G	5				
							2	95.84
- stiff by 2.5m depth			G	6				
	3.35						3	94.84
End of Test Pit								
(Groundwater infiltration at 1.8m depth)								



Piezometer Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - Wateridge Block 29
Burma Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant along Wanaki Road. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

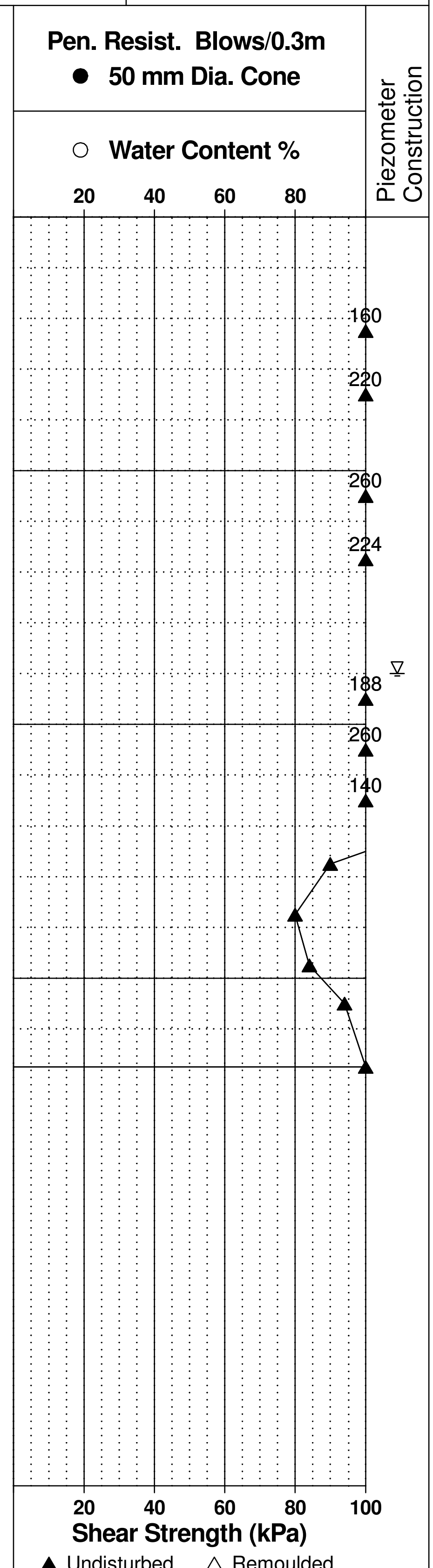
BORINGS BY Hydraulic Shovel

DATE 2019 May 6

FILE NO. PG4965

HOLE NO. TP 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.71
FILL: Silty clay with gravel 0.10							
TOPSOIL 0.35		G	1				
		G	3				
		G	4			1	96.71
Hard to very stiff, grey-brown **SILTY CLAY**							
		G	5				
		G	6			2	95.71
- stiff and grey by 2.5m depth							
		G	7			3	94.71
3.35							
End of Test Pit							
(Groundwater infiltration at 1.8m depth)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - Wateridge Block 29
Burma Road, Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant along Wanaki Road. An arbitrary elevation of 100.00m was assigned to the TBM.

REMARKS

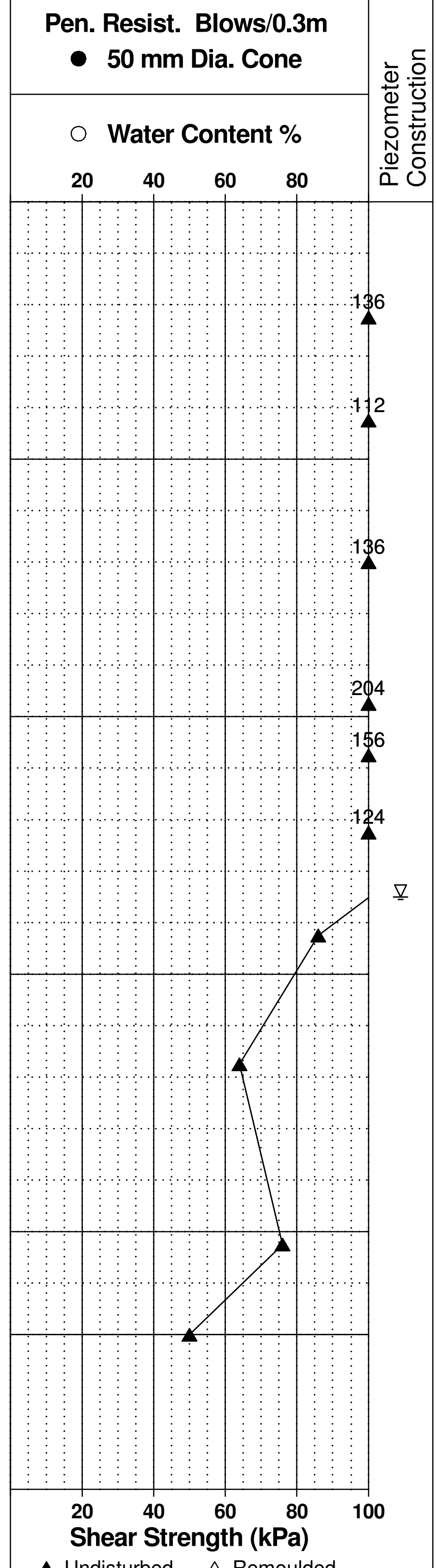
BORINGS BY Hydraulic Shovel

DATE 2019 May 6

FILE NO. PG4965

HOLE NO. TP 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.81
FILL: Crushed stone		G	1				
0.35							
Very stiff to stiff, grey-brown **SILTY CLAY**		G	2				
		G	3				
						1	96.81
- grey by 2.1m depth		G	4				
						2	95.81
		G	5				
		G	6				
						3	94.81
						4	93.81
4.40							
End of Test Pit							
(Groundwater infiltration at 2.7m depth)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

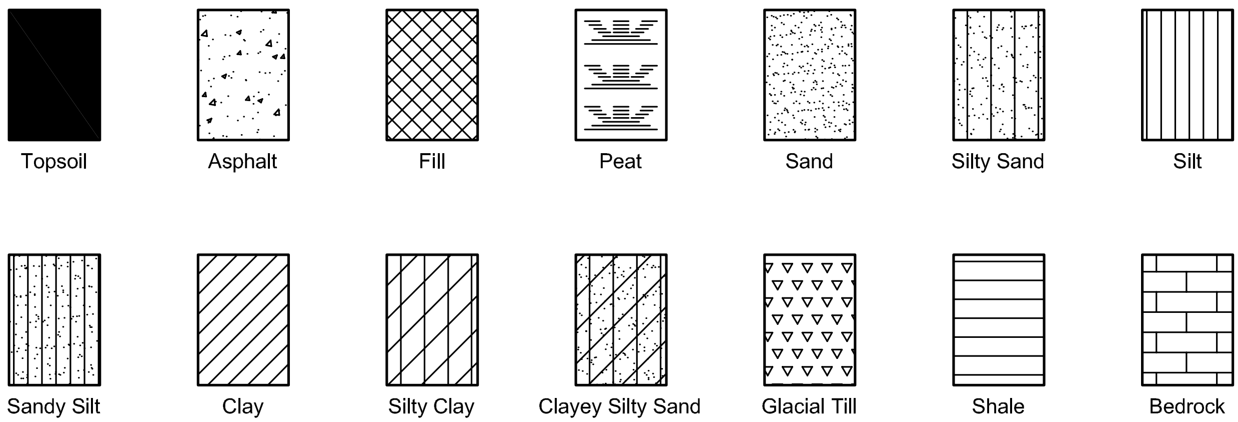
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

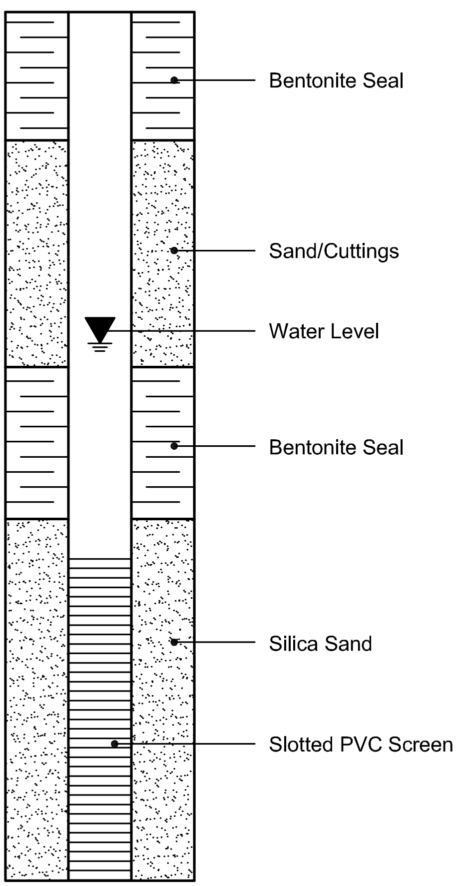
SYMBOLS AND TERMS (continued)

STRATA PLOT

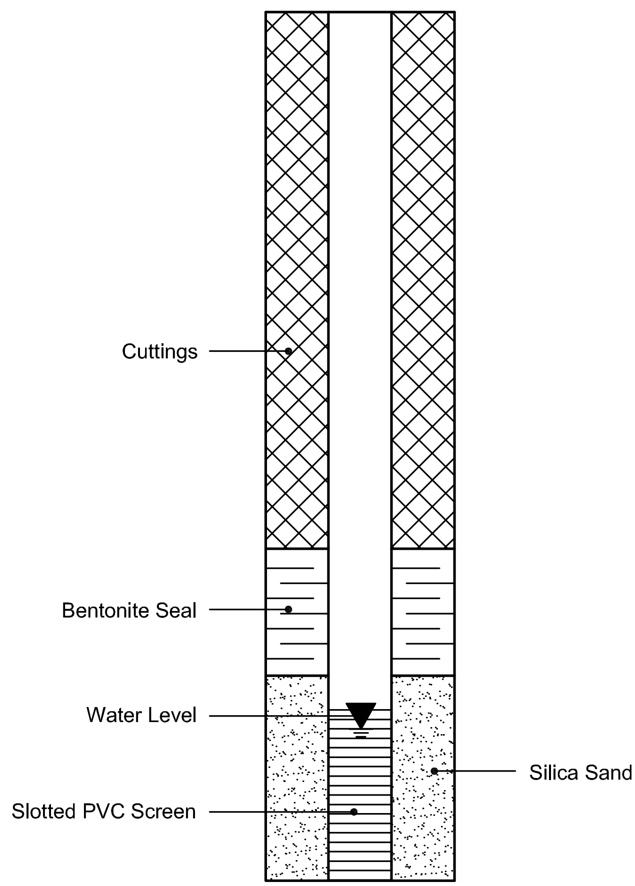


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 26889

Report Date: 14-Jun-2019

Order Date: 10-Jun-2019

Project Description: PG4965

	Client ID:	TP3-G4	-	-	-
	Sample Date:	07-Jun-19 14:00	-	-	-
	Sample ID:	1924101-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	70.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.22	-	-	-
Resistivity	0.10 Ohm.m	38.5	-	-	-
Anions					
Chloride	5 ug/g dry	78	-	-	-
Sulphate	5 ug/g dry	278	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4965-1 - TEST HOLE LOCATION PLAN

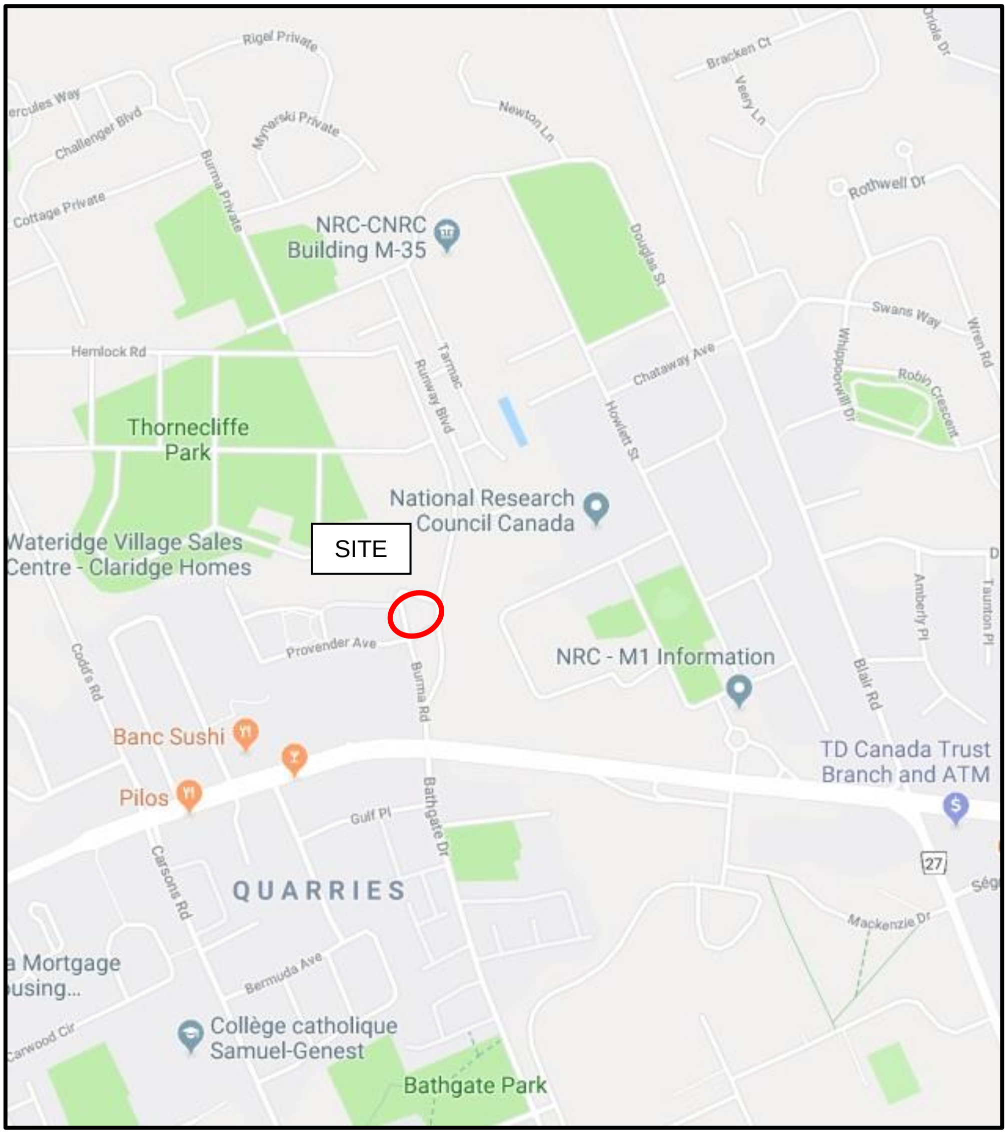
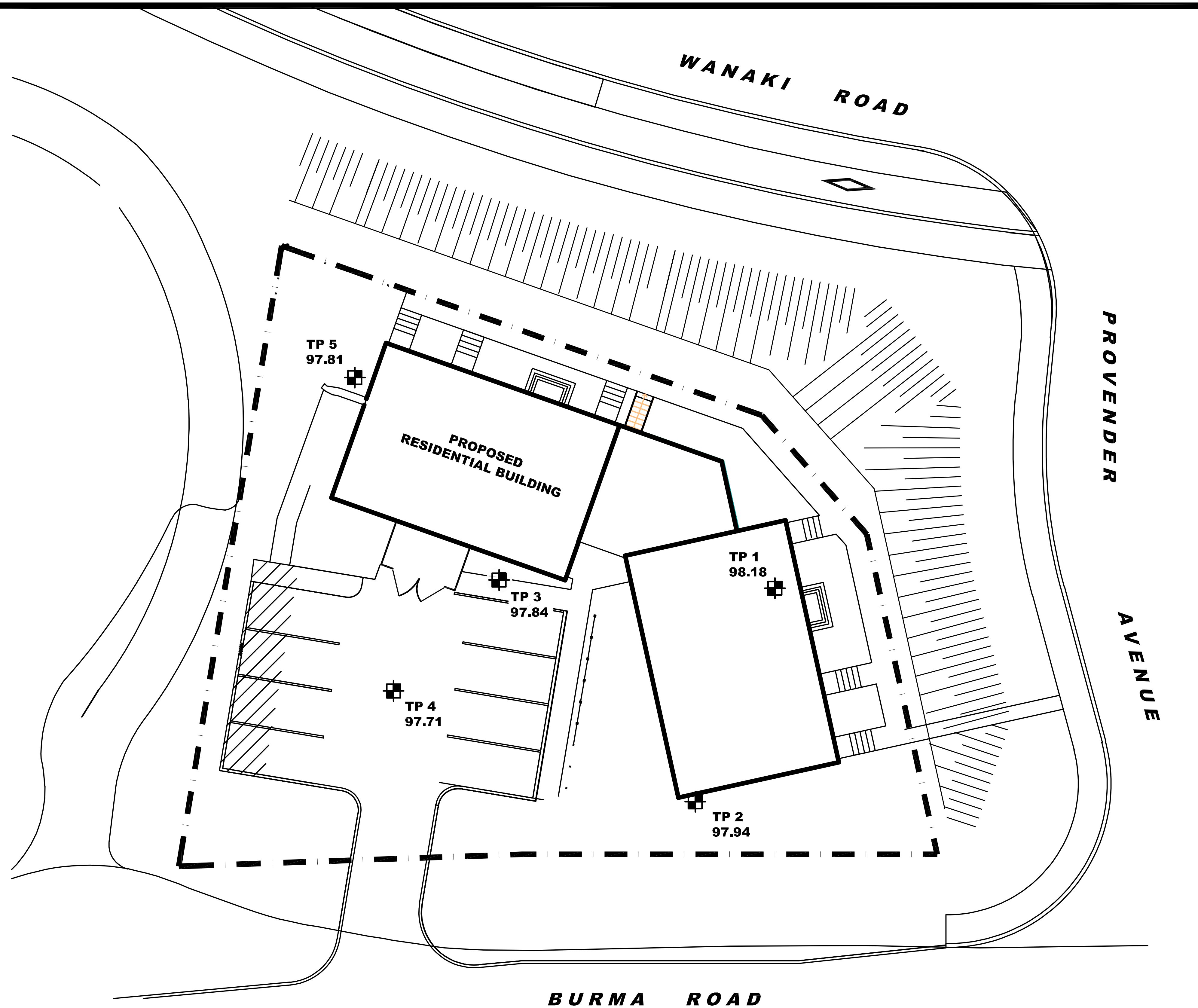


FIGURE 1

KEY PLAN



LEGEND:

TEST PIT LOCATION

98.18 GROUND SURFACE ELEVATION (m)

TBM - TOP SPINDLE OF FIRE HYDRANT ALONG WANAKI ROAD. AN ARBITRARY ELEVATION OF 100.00m WAS ASSIGNED TO THE TBM.

BASE PLAN PROVIDED BY CSV ARCHITECTS.

SCALE: 1:250

0 1 2 3 4 5 10 15m

p:\autocad drawings\geotechnical\pg49xx\pg4965-1 thlp.dwg

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

COLONNADE BRIDGEPORT

GEOTECHNICAL INVESTIGATION
PROP. RESIDENTIAL BUILDING - WATERIDGE BLOCK 9 - BURMA ROAD
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 06/2019
Drawn by: MPG	Report No.: PG4965-1
Checked by: DJG	**PG4965-1**
Approved by: DJG	Revision No.: