

Geotechnical
Engineering

Environmental
Engineering

Hydrogeology

Geological
Engineering

Materials Testing

Building Science

Archaeological Services

Geotechnical Investigation

Proposed Residential Development
73 Guigues Avenue
Ottawa, Ontario

Prepared For

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c/o Kasuco Mortgages & Investments Inc.

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Report PG4601-1 Revision 1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by 1470087 Ontario Inc. to conduct a geotechnical investigation for the proposed residential development to be located at 73 Guigues Avenue, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the investigation were 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 the design.

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

2.0 Proposed Project

Based on preliminary design details, it is understood that the proposed development consists of eight (8) 3-storey slab-on-grade townhouses with associated access lanes and landscaped areas. It is expected that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was conducted on August 1, 2018. At that time, a total of three (3) boreholes were drilled to a maximum depth of 1.5 m below existing ground surface. The test hole locations were selected in a manner to provide general coverage of the proposed development. The test hole locations are provided on Drawing PG4601-1-Test Hole Location Plan included in Appendix 2.

The test holes were advanced with a truck-mounted 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 geotechnical 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 soil samples were transported to our laboratory. The depths at which the auger and split spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets 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.

Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

Groundwater

Flexible polytubing was installed in the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

3.2 Field Survey

The borehole locations were laid out in the field and surveyed by Paterson. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), consisting of the top of spindle of the fire hydrant located across Parent Avenue from the subject site. An arbitrary elevation of 100.00 m was assigned to the TBM. The location of the TBM and ground surface elevations at the borehole locations are presented on Drawing PG4601-1 - 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. Soil samples will be stored for a period of one month after this report is completed, unless otherwise directed.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by several residential buildings and an associated asphalt surfaced parking lot. It is anticipated that the existing buildings will be demolished as part of the subject development.

The subject site is relatively flat and at-grade with the surrounding properties and roadways. Surface drainage is provided by catch basins located on the adjacent roadway. The site is bordered by residential buildings to the east, St. Andrew Street to the north, Parent Avenue to the west and Guigues Avenue to the south.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the test hole locations consists of an asphalt pavement structure overlying a fill layer consisting of hard silty clay with sand and gravel which extends to a depth of approximately 1.1 to 1.5 m. Practical refusal to augering was encountered immediately below the fill layer. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Based on available geological mapping, the local bedrock consists of interbedded limestone and shale of the Verulam Formation with an anticipated overburden thickness of 1 to 3 m.

4.3 Groundwater

Groundwater levels were recorded at the piezometers installed at the borehole locations on August 13, 2018. Groundwater was not observed in any of the piezometers at the time of recording. It should be noted that groundwater levels are subject to seasonal fluctuations and therefore levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed development. The proposed building is expected to be founded on conventional footings placed on clean, surface sounded bedrock. Bedrock removal may be required for this project to achieve required footing depths.

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

5.2 Site 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 is anticipated that the existing fill, free of deleterious material and significant amounts of organics, can be left in place below the proposed building footprints, outside of lateral support zones for the footings. However, it is recommended that the existing fill layer be proof-rolled several times and approved by Paterson at the time of construction. Any poor-performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill material.

Bedrock Removal

Based on the bedrock encountered in the area, it is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the effects on the existing services, building and other structures should be addressed. A pre-blast or construction survey located in proximity of the blasting operations should be conducted prior to commencing construction. The extent of the survey should be determined by the blasting consultant and sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

Excavation side slopes in sound bedrock could be completed with nearly vertical side walls. Where bedrock is of lower quality, the excavation face should be free of any loose rock. An area specific review should be completed by the geotechnical consultant at the time of construction to determine if rock bolting or other remedial measures are required to provide a safe excavation face for areas where low quality bedrock is encountered.

Vibration Considerations

Construction operations could cause vibrations, and possibly sources of nuisance to the community. Therefore, means to reduce the vibration levels and much as possible should be incorporated in the construction operations to maintain a cooperative environment with the residents.

The following construction equipment could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system with soldier piles, should it be utilized, would require these pieces of equipment. Vibrations, caused by blasting or construction operations could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the recommended vibration limit, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people. Therefore, a pre-construction survey is recommended to minimize the risks of claims or following the construction of the proposed building.

Fill Placement

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

The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the proposed building areas 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 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 their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface-sounded bedrock bearing surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near-surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

In areas where the bedrock elevation is deeper than design underside of footing elevation, footings may be placed on existing fill or an engineered fill pad placed over existing fill or bedrock. The existing fill or engineered fill pad must be approved by Paterson at the time of construction.

Footings bearing on existing fill, free of any significant amount of organic or otherwise deleterious materials, proof-rolled using suitable vibratory compaction equipment making several passes under dry conditions and above freezing temperatures, and approved by Paterson personnel can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ULS of **150 kPa**, incorporating a geotechnical resistance factor of 0.5.

Footings bearing on an engineered fill pad, placed over bedrock or an existing fill subgrade approved by Paterson at the time of construction, can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa**, incorporating a geotechnical resistance factor of 0.5. The engineered fill should consist of OPSS Granular A or B Type II, placed in maximum 300 mm thick loosed lifts and compacted to minimum 98% SPMDD.

Footings bearing on approved existing fill or engineered fill pad and designed using the above noted bearing resistance values will be subjected to post-construction total and differential settlements of 25 mm 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 sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Footings bearing on approved existing fill or an engineered fill pad will require a lateral support zone of 1.5H:1V (or flatter).

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the shallow foundations considered at the subject site. A higher site class, such as Class A or B, may be provided for foundations placed on or near the bedrock surface. However, the higher site class would have to be confirmed by site specific seismic shear wave velocity testing. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code (OBC) 2012 for a full discussion of earthquake design requirements.

5.5 Slab on Grade Construction

With the removal of all topsoil and deleterious fill, such as those containing significant amounts of organic matter and other deleterious material within the footprint of the proposed buildings, engineered fill or existing fill approved by Paterson at the time of construction will be considered an acceptable subgrade on which to commence backfilling for floor slab construction. It is recommended that the existing fill layer, free of deleterious and organic materials, be proof-rolled several times and approved by Paterson at the time of construction. Any soft areas should be removed and backfilled with appropriate granular material. It is recommended that the upper 200 mm of sub-slab fill consist of OPSS Granular A crushed stone.

5.6 Pavement Structure

For design purposes, the pavement structures presented in the following tables are recommended for the design of car only parking areas and access lanes.

Table 1 - Recommended Flexible 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 Flexible Pavement Structure - Access Lanes	
Thickness (mm)	Material Description
40	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course - HL-8 or Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
450	SUBBASE - OPSS Granular B Type II
	SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill

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

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

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.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended to be provided for the proposed building to provide an outlet for surface water trapped within the backfill material below any sidewalk structure. Trapped water within the subgrade soils can lead to more significant frost heave for sidewalks adjacent to slab-on-grade buildings. The system should consist of a 150 mm diameter, geotextile wrapped, perforated corrugated plastic pipe, surrounding on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the buildings. 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 greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

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

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the 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 side slopes of excavations in the soil and fill overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is anticipated that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should 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.

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 should not remain open for extended periods of time.

In bedrock, almost vertical side slopes can be constructed, provided all weathered and loose rock is removed or stabilized with rock anchors as per Paterson recommendations.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements should be designed by a structural engineer, specializing in shoring design. The shoring will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations, roadways and underground services.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

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

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

Table 4 - Soil Parameters	
Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m ³	23.5
Effective Unit Weight (γ), kN/m ³	13.7

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

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 Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 300 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on bedrock subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the pipe obvert 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% SPMDD.

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 reduce potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

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.

Infiltration levels are anticipated to be low through the excavation face. The groundwater infiltration will be controllable with open sumps and pumps.

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

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

Long-Term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building's perimeter or sub-slab drainage system will be directed to the storm sewer. It is expected that groundwater flow will be low (i.e. less than 25,000 L/day) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed within the excavation. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

Based on our observations, a local groundwater lowering is not anticipated under short-term conditions due to construction of the proposed building. The neighbouring structures are expected to be founded directly over a bedrock bearing surface. Issues are not expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions (in particular, where a shoring system is constructed), the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

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.

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

6.7 Corrosion Potential and Sulphate

The results of the analytical testing show that the sulphate content is less than 0.1%. These results are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The results of the chloride and sulphate content, pH and resistivity indicate the presence of a moderately aggressive to very aggressive environment for exposed ferrous metals at this site.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should 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, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Kasuco Mortgages & Investments Inc. or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.



Nathan F. S. Christie, P.Eng.



David J. Gilbert, P.Eng.

Report Distribution:

- Kasuco Mortgages & Investments Inc. (3 copies)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
 Prop. Residential Development - 73 Guigues Avenue
 Ottawa, Ontario

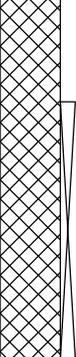
DATUM TBM - Top spindle of fire hydrant located near the northwest corner of Parent Avenue and Guigues Avenue. An arbitrary elevation of 100.00m was assigned to the TBM.

FILE NO.
PG4601

HOLE NO.
BH 1

BORINGS BY CME 55 Power Auger

DATE August 1, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
GROUND SURFACE								20	40	60	80	
25mm Asphaltic concrete		AU	1			0	98.11					
FILL: Brown silty clay with sand, gravel, cobbles and boulders		SS	2	67	50+	1	97.11					
End of Borehole	1.14											
Practical refusal to augering at 1.14m depth (BH dry upon completion)												

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
 Prop. Residential Development - 73 Guigues Avenue
 Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the northwest corner of Parent Avenue and Guigues Avenue. An arbitrary elevation of 100.00m was assigned to the TBM.

FILE NO.
PG4601

HOLE NO.
BH 2

BORINGS BY CME 55 Power Auger

DATE August 1, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
GROUND SURFACE								20	40	60	80	
30mm Asphaltic concrete		AU	1			0	98.40					
FILL: Brown silty clay with sand grave, cobbles and boulders		SS	2		57	1	97.40					
End of Borehole	1.47											
Practical refusal to augering at 1.47m depth (BH dry upon completion)												

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
 Prop. Residential Development - 73 Guigues Avenue
 Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located near the northwest corner of Parent Avenue and Guigues Avenue. An arbitrary elevation of 100.00m was assigned to the TBM.

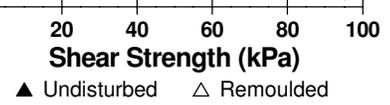
FILE NO.
PG4601

HOLE NO.
BH 3

BORINGS BY CME 55 Power Auger

DATE August 1, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
GROUND SURFACE								20	40	60	80	
25mm Asphaltic concrete		AU	1			0	98.65					
FILL: Brown silty clay with sand, gravel, cobbles and boulders		SS	2	0	50+	1	97.65					
End of Borehole	1.47											
Practical refusal to augering at 1.47m depth (BH dry upon completion)												



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)
D _{xx}	-	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
D ₁₀	-	Grain size at which 10% of the soil is finer (effective grain size)
D ₆₀	-	Grain size at which 60% of the soil is finer
C _c	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
C _u	-	Uniformity coefficient = D_{60} / D_{10}

C_c and C_u are used to assess the grading of sands and gravels:

Well-graded gravels have: $1 < C_c < 3$ and $C_u > 4$

Well-graded sands have: $1 < C_c < 3$ and $C_u > 6$

Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.

C_c and C_u 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
C _{cr}	-	Recompression index (in effect at pressures below p' _c)
C _c	-	Compression index (in effect at pressures above p' _c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
W _o	-	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.
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SYMBOLS AND TERMS (continued)

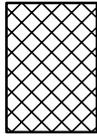
STRATA PLOT



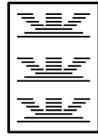
Topsoil



Asphalt



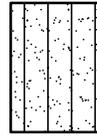
Fill



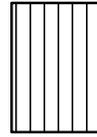
Peat



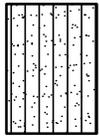
Sand



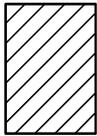
Silty Sand



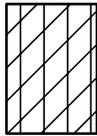
Silt



Sandy Silt



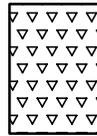
Clay



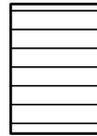
Silty Clay



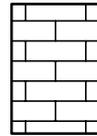
Clayey Silty Sand



Glacial Till



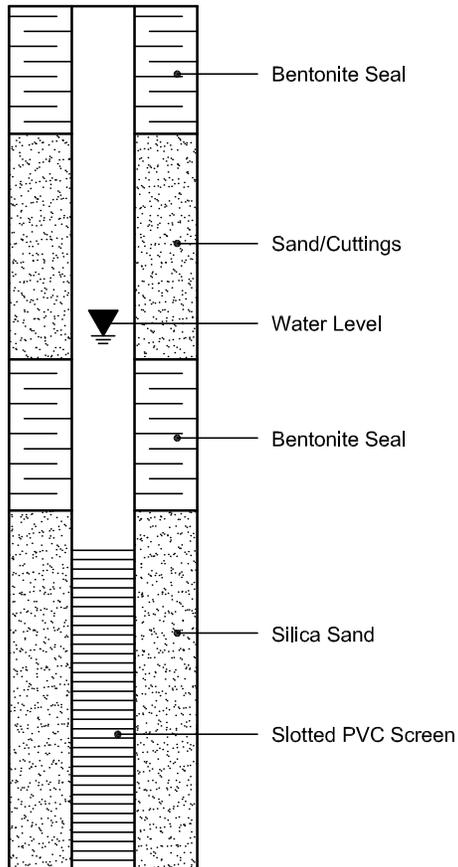
Shale



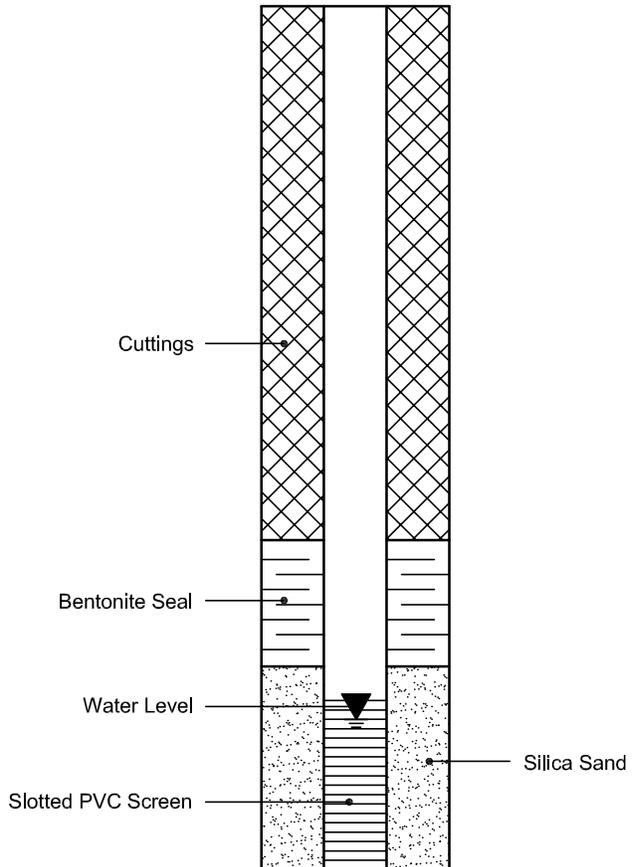
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis
 Client: Paterson Group Consulting Engineers
 Client PO: 24855

Report Date: 08-Aug-2018

Order Date: 3-Aug-2018

Project Description: PG4601

Client ID:	BH2 SS2	-	-	-
Sample Date:	01-Aug-18	-	-	-
Sample ID:	1831493-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	96.1	-	-	-
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General Inorganics

pH	0.05 pH Units	7.52	-	-	-
Resistivity	0.10 Ohm.m	28.2	-	-	-

Anions

Chloride	5 ug/g dry	57	-	-	-
Sulphate	5 ug/g dry	50	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4601-1 - TEST HOLE LOCATION PLAN

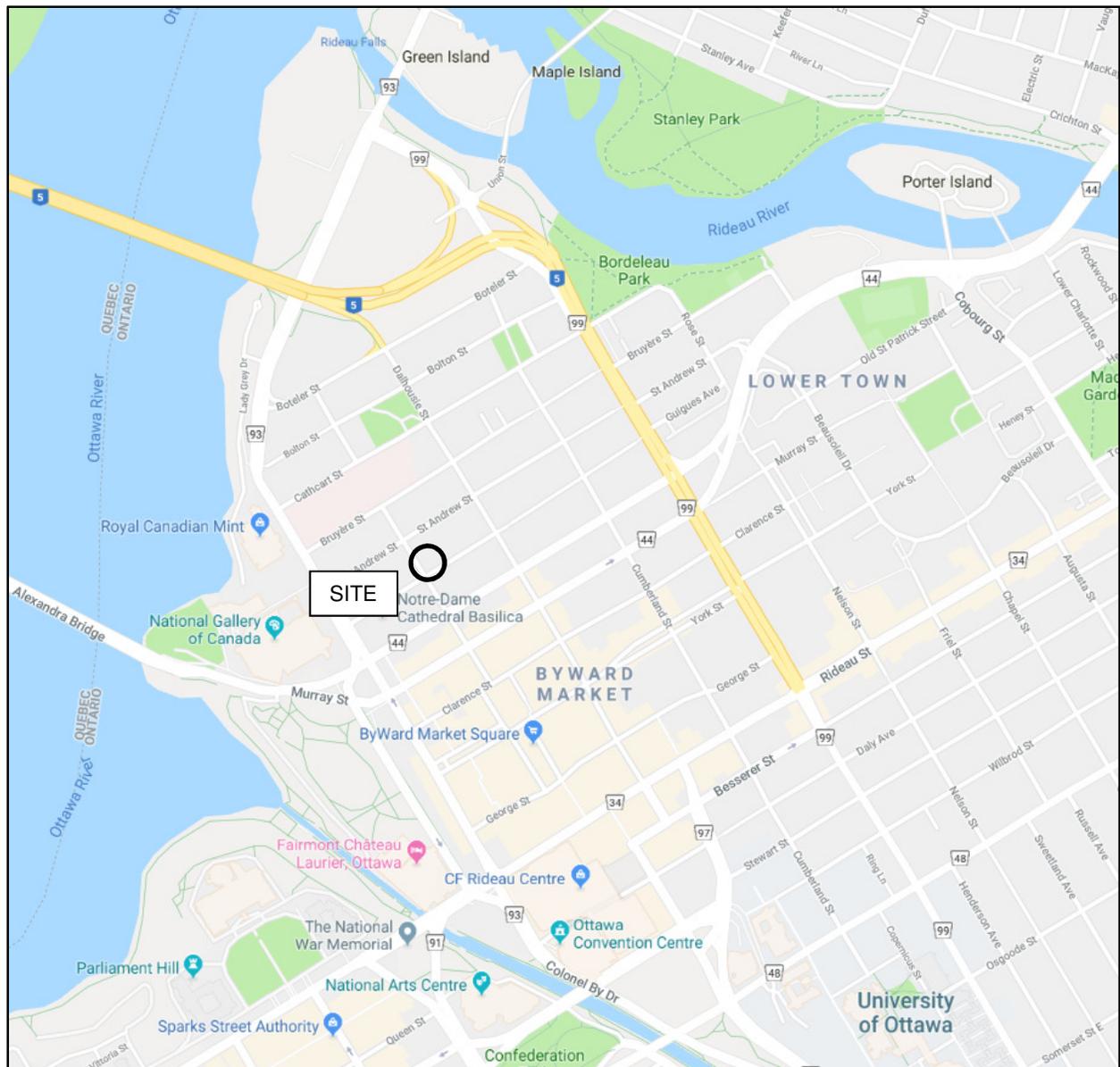
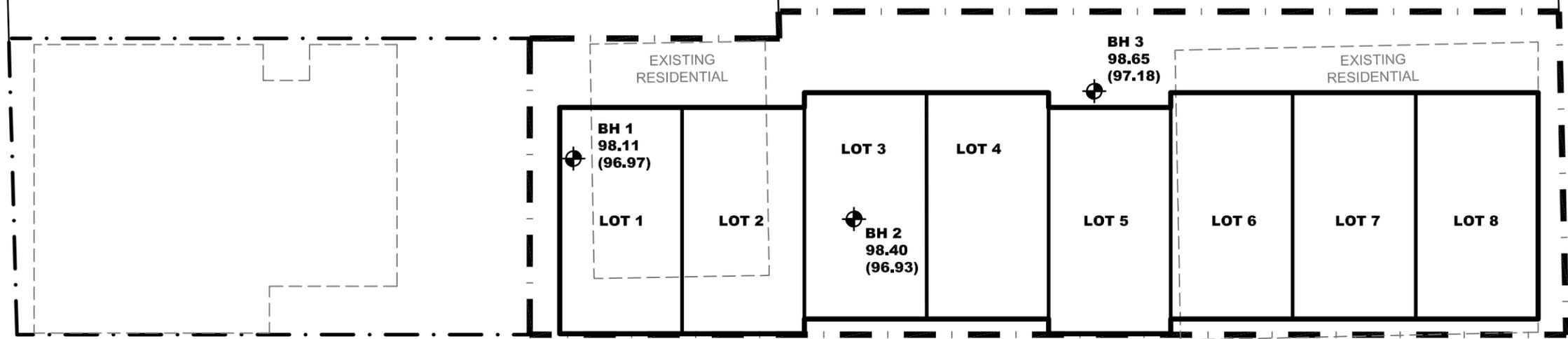


FIGURE 1

KEY PLAN

ST. ANDREW STREET

GUIGUES AVENUE



P A R E N T A V E N U E

LEGEND:

-  BOREHOLE LOCATION
- 98.11 GROUND SURFACE ELEVATION (m)
- (96.97) PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

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



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NO.	REVISIONS	DATE	INITIAL
0			

1470087 ONTARIO INC.
GEOTECHNICAL INVESTIGATION
 PROP. RESIDENTIAL DEVELOPMENT - 73 GUIGUES AVENUE
 OTTAWA, ONTARIO

Title: **TEST HOLE LOCATION PLAN**

Scale:	1:200	Date:	08/2018
Drawn by:	MPG	Report No.:	PG4601-1
Checked by:	NC	Dwg. No.:	PG4601-1
Approved by:	DJG	Revision No.:	0

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