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Geotechnical Engineering

Environmental Engineering

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Materials Testing

Building Science

Archaeological Services

Geotechnical Investigation
Proposed Multi-Storey Residential Buildings
3484, 3490 and 3592 Innes Road
Ottawa, Ontario

Prepared For

Canadian Rental Development Services Inc.

May 21, 2019

Report: PG4488-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Canadian Rental Development Services Inc. to conduct a geotechnical investigation for the proposed multi-storey residential buildings to be located at 3484, 3490 and 3592 Innes Road, in the City of Ottawa, Ontario (Refer to Figure 1 - Key Plan in Appendix 2)

The objectives of the current desktop review were:

- ❏ to determine the subsurface soil and groundwater conditions based on the existing soils information,
- ❏ to provide geotechnical recommendations pertaining to 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 the geotechnical findings and recommendations pertaining to the design and construction of the subject development as understood at the time of writing this report.

2.0 Proposed Development

It is understood that the proposed development will consist of 8 multi-storey buildings with one shared underground parking level occupying the majority of the subject site. It should be noted that an at-grade parking level is proposed above the underground structure to extend between the proposed buildings. Associated access lanes and landscaped areas are also anticipated for the subject development. It is further understood that the site will be municipally serviced.

3.0 Existing Information

An existing geotechnical investigation was completed by others in 2016. At that time, a total of 7 boreholes were advanced to a maximum depth of 7 m below existing ground surface. The locations of the test holes are presented in the Drawing PG4488-1 - Test Hole Location Plan in Appendix 2.

Overburden (North Portion)

Generally, the subsurface profile at the test hole locations along the north portion of the site consists of a fill layer of brown sandy silt to clayey silt with rootlets, overlying a very stiff brown-grey silty clay layer. Glacial till was encountered below the above noted soils consisting of silty sand with gravel, cobbles and boulders. Practical refusal to augering was encountered below the glacial till layer in all borehole locations. It should be noted that bedrock was encountered between 0.9 to 5 m below existing grade within the north portion of the site.

Overburden (South Portion)

Generally, the subsurface profile at the test hole locations along the south portion of the site consists of a very stiff to stiff brown silty clay crust overlying a soft to stiff grey silty clay layer. A thin layer of glacial till was encountered below the above noted layers consisting of silty sand with gravel, cobbles and boulders. Practical refusal to augering was encountered below the glacial till layer. It should be noted that bedrock was encountered between 5.6 to 7 m below existing grade within the south portion of the site.

The test hole logs completed by others for the subject site are presented in Appendix 1 of this report.

Bedrock

Based on available geological mapping, the local bedrock consists of limestone and shale of the Bobcaygeon and Lindsay formations and the overburden thickness varies between 0 to 10 m.

Groundwater

Based on available soils information, it is estimated that the long-term groundwater level can be expected between 2.5 to 3.5 m below existing ground surface. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

4.0 Supplemental Investigation

4.1 Field Program

The field program for the current investigation was carried out June 13 and 14, 2018. At that time, 102 probe holes were advanced to a maximum depth of 11.2 m or refusal over bedrock surface. The locations of the probe holes are presented in Drawing PG4488-1 - Test Hole Location Plan in Appendix 2.

The probe holes were completed using a track mounted air-track drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer. The bedrock depth is also inferred based on the results if the probe hole investigation.

4.2 Field Survey

The test hole locations were determined by Paterson field personnel. Test hole locations and ground surface elevations at the test hole locations were provided by Annis O'Sullivan Vollebekk Ltd. It is understood that the test hole locations and ground surface elevations are referenced to a geodetic datum. The location of the boreholes and the ground surface elevation at each borehole location are presented on Drawing PG4488-1 - Test Hole Location Plan in Appendix 2.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered adequate for the proposed development. The proposed multi-storey buildings can be founded over conventional style shallow foundations placed on undisturbed, stiff to firm silty clay, glacial till or a clean, surface sounded bedrock bearing surface. Buildings placed within the south portion of the subject site may require a series of near vertical, zero entry, concrete in-filled trenches extending to a clean, surface-sounded bedrock surface due to the depth of the clay deposit which may extend below design underside of footing level.

If buildings are founded directly over a clay deposit, a permissible grade raise restriction will be required. A preliminary permissible grade raise restriction of **2 m** is recommended for the south portion of the site.

If bedrock removal is required, consideration should be given to hoe-ramming or controlled blasting. 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 blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

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

The above and other considerations are 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.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or hoe ramming.

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

As a general guideline, peak particle velocities (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 also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be excavated almost vertical side walls. A minimum 1 m horizontal ledge, should remain between the overburden excavation and the bedrock surface. The ledge will provide an area to allow for potential sloughing or a stable base for the overburden shoring system.

Horizontal Rock Anchors

Bedrock stabilization may be required when the proposed foundation extends into the bedrock.

Rock anchors and rock face protection may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface.

The requirement for rock face protection and rock anchors should be evaluated during the excavation operations and should be discussed with the structural engineer during the design stage.

Fill Placement

Fill placed for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick lifts and compacted to 98% of standard Proctor maximum dry density (SPMDD). Site crushed bedrock can also be considered for placement below the building area. It is recommended that a well graded material with a maximum diameter of 300 mm be used.

5.3 Foundation Design

Bearing Resistance Values (Conventional Shallow Foundation)

Bedrock Medium

Footings placed on a clean, surface sounded bedrock surface can be designed using a bearing resistance value at ULS of **3,000 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS. Alternatively, footings placed over zero entry, near vertical trenches extending to bedrock and in-filled with lean concrete (15 MPa) to underside of footing level can be designed using the values provided above. It is recommended that the trench sidewalls extend at least 300 mm beyond the outside face of the footings.

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 surface sounded bedrock and designed using the above noted bearing resistance values will be subjected to negligible post-construction total and differential settlements.

Overburden

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on an undisturbed, stiff silty clay can be designed using the bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**.

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on an undisturbed, soft to firm silty clay can be designed using the bearing resistance value at serviceability limit states (SLS) of **75 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **125 kPa**

Footings placed on an undisturbed, compact silty sand, glacial till, or engineered fill bearing surface can be designed using a bearing resistance value at SLS of **150 kPa** and at ULS of **225 kPa**. A geotechnical resistance factor of 0.5 was incorporated into the above noted bearing resistance values at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed, in the dry, prior to the placement of concrete for footings.

Footings bearing on an undisturbed soil bearing surface and designed using the bearing resistance values provided herein 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 an undisturbed soil bearing surface above the groundwater table when a plane extending horizontally and vertically from the bottom edge of the footing at a minimum of 1.5H:1V, passing through in situ soil of the same or higher capacity as the bearing medium soil.

Bedrock/Soil Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long term total and differential settlements. Also, at the soil/bedrock and bedrock/soil transitions, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material. The width of the subexcavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

Lean Concrete Filled Trenches

Where bedrock is encountered below the design underside of footing elevation, consideration should be given to excavating vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (**15 MPa** 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, an assessment should be completed to determine the water infiltration and stability of the excavation sidewalls extending to the bedrock surface.

The trench excavation should be at least 150 mm wider than all sides of the footing at the base of the excavation. The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a 1.5 m depth). Once approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

Footings placed on lean concrete filled trenches extending to the bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **3,000 kPa**.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for foundations to be constructed within the subject site. A higher seismic site classification such as Class A or B can be used for the subject site provided a site specific shear wave velocity test be completed. The soils underlying the proposed shallow foundations are not susceptible to liquefaction. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Slab

The basement area for the proposed project will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be constructed, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. The upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone for slab on grade construction. All backfill material within the footprint of the proposed building(s) should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD. Alternatively, where the depth of in-filling is greater than 600 mm, blast rock with a maximum particle size no greater than 300 mm diameter can be used below the floor slab. The blast rock should be placed in maximum 300 mm loose lifts and compacted using vibratory compaction equipment making several passes. The compaction efforts should be completed under dry conditions and above freezing temperatures and be approved by Paterson personnel at the time of construction.

In consideration of the groundwater conditions encountered at the time of the current and previous fieldwork, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor (discussed in Subsection 6.1).

5.6 Basement Walls

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structures. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. The applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (P_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil, 0.5
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the "at-rest" case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

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

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

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

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

5.7 Pavement Structure

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

Table 1 - Recommended Flexible Pavement Structure - Parking Level

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, OPSS Granular B Type II material placed over in situ soil or fill	

Table 2 - Recommended Flexible 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 or OPSS Granular B Type I or II material placed over in situ soil or fill	

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

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

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the SPMDD.

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 structures. The composite drainage system (such as Miradrain G100N, Delta Drain 6000 or an approved equivalent) is recommended to extend to the footing level. Sleeves, 150 mm diameter, at 3 m centres are recommended to be placed in the footing or at the foundation wall/footing interface for blind sided pours to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

Underfloor drainage is recommend to control water infiltration due to groundwater infiltration at the proposed founding elevation. For design purposes, Paterson recommends 150 mm diameter perforated pipes be placed between each parking bay. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

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 where frost heave sensitive structures, unless a composite drainage system is in place. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material may be used for this purpose. A composite drainage system, such as Delta Drain 6000, Miradrain G100 or equivalent, should be placed against the foundation wall to promote drainage toward the perimeter drainage pipe.

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 of 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.

Unheated structures, such as access ramps to underground parking, may be required to be insulated against the deleterious effects of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation should be provided.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structures are backfilled.

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.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's structural design prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

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

Table 3 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

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

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.

Underpinning

Founding conditions of adjacent structures bordering the site should be assessed and underpinning requirements should be evaluated.

6.4 Pipe Bedding and Backfill

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 silty clay, the thickness of the bedding material may require to be increased to a minimum of 300 mm. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

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

Generally, it should be possible to re-use the moist, not wet, silty clay and silty sand above the cover material if the excavation and filling operations are carried out in dry weather conditions.

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.

Clay Seals

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

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

A temporary Ministry of the Environment, Conservation and Parks (MECP) 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.

Impacts on Neighbouring Structures

Based on observations, the groundwater level is anticipated at a 2.5 to 3.5 m depth. A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed buildings. The extent of any significant groundwater lowering should occur within a limited range of the subject site due to the minimal temporary groundwater lowering. No issues are expected, with respect to groundwater lowering, that would cause long term damage to adjacent structures surrounding the proposed buildings.

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 buildings 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.

7.0 Recommendations

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

☐ Review the bedrock stabilization and excavation requirements.

☐ Review proposed waterproofing and foundation drainage design and requirements.

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

☐ Sampling and testing of the concrete and fill materials.

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

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

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

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

8.0 Statement of Limitations

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

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, we request that we be notified 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 Canadian Rental Development Services Inc. 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.

Faisal I. Abou-Seido, P.Eng.

LICENSED PROFESSIONAL ENGINEER
May 21, 2019
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PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

Report Distribution:

☐ Canadian Rental Development Services Inc. (3 copies)
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APPENDIX 1

TEST HOLE LOGS - BY OTHERS

PROJECT: 1660030

RECORD OF BOREHOLE: 16-1

SHEET 1 OF 1

LOCATION: N 5034598.0 ;E 381071.2

BORING DATE: November 1, 2016

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES		
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m
		GROUND SURFACE		90.95			
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE *(60 mm)*		0.06			
		FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE)		90.68 0.33			
		ASPHALTIC CONCRETE *(50 mm)*					
		FILL - (SM) SILTY SAND, trace gravel and organics; brown to black; non-cohesive, moist, loose		89.88 1.07	1	SS	9
1		FILL - (SM/ML) SILTY SAND to sandy SILT, some gravel; brown; non-cohesive, moist, loose		89.43 1.52			
		(SM) gravelly SILTY SAND; brown, contains cobbles and boulders (GLACIAL TILL); non-cohesive, moist		89.12 1.83	2	SS	>50
2		End of Borehole Auger Refusal					
3							
4							
5							
6							
7							
8							
9							
10							

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
20 40 60 80

SHEAR STRENGTH Cu, kPa
nat V. + Q - ●
rem V. ⊕ U - ○
20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s
10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT
Wp ├──○W──┤ Wl
20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

MIS-BHS 001 1660030-GEOTECH.GPJ GAL-MIS.GDT 12/16/16 JEM

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: DWM
CHECKED: WAM

PROJECT: 1660030

RECORD OF BOREHOLE: 16-2

SHEET 1 OF 1

LOCATION: N 5034649.4 ;E 381258.4

BORING DATE: November 1, 2016

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m	SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	WATER CONTENT PERCENT Wp ⊢ W ○ ⊣ Wl 20 40 60 80		
		GROUND SURFACE		90.23							
0	Power Auger 200 mm Diam. (HS)	FILL - (SP) gravelly SAND, some non-plastic fines; grey brown; non-cohesive, moist, compact	▨	0.00							
		FILL - (SM/SC) SILTY SAND to CLAYEY SAND, trace gravel; grey brown; non-cohesive, moist	▨	89.93 0.30	1	SS	26				
				89.32	2	SS	>50				
1		End of Borehole Auger Refusal		0.91							
2											
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: DWM

CHECKED: WAM

MIS-BHS 001 1660030-GEOTECH.GPJ GAL-MIS.GDT 12/16/16 JEM

PROJECT: 1660030

RECORD OF BOREHOLE: 16-2A

SHEET 1 OF 1

LOCATION: 1.8 m East of BH 16-2

BORING DATE: November 1, 2016

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		90.23							
0	Power Auger 200 mm Diam. (Hollow Stem)	For Stratigraphy see RECORD OF BOREHOLE 16-2		0.00							
1		End of Borehole Auger Refusal		89.26 0.97							
2											
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: DWM

CHECKED: WAM

PROJECT: 1660030

RECORD OF BOREHOLE: 16-3

SHEET 1 OF 1

LOCATION: N 5034442.3 ;E 381033.5

BORING DATE: November 1, 2016

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m
GROUND SURFACE	88.98			
FILL/TOPSOIL - (SM) SILTY SAND; brown	0.00			
FILL - (ML) CLAYEY SILT, some sand; brown, contains roots (reworked native soil); cohesive, w>PL	0.15	1	SS	2
(CI/CH) SILTY CLAY to CLAY, trace sand; brown (WEATHERED CRUST); cohesive, w>PL, very stiff to stiff	88.37 0.61	2 3	SS SS	8 4
(CI/CH) SILTY CLAY to CLAY; grey with brown mottling, contains occasional white shell; cohesive, w>PL, soft to firm	85.93 3.05	4 5	SS SS	PH WH
(ML) sandy CLAYEY SILT, trace gravel; grey, contains cobbles and boulders (GLACIAL TILL); non-cohesive, wet	83.95 5.03			
End of Borehole Auger Refusal	83.65 5.33			

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

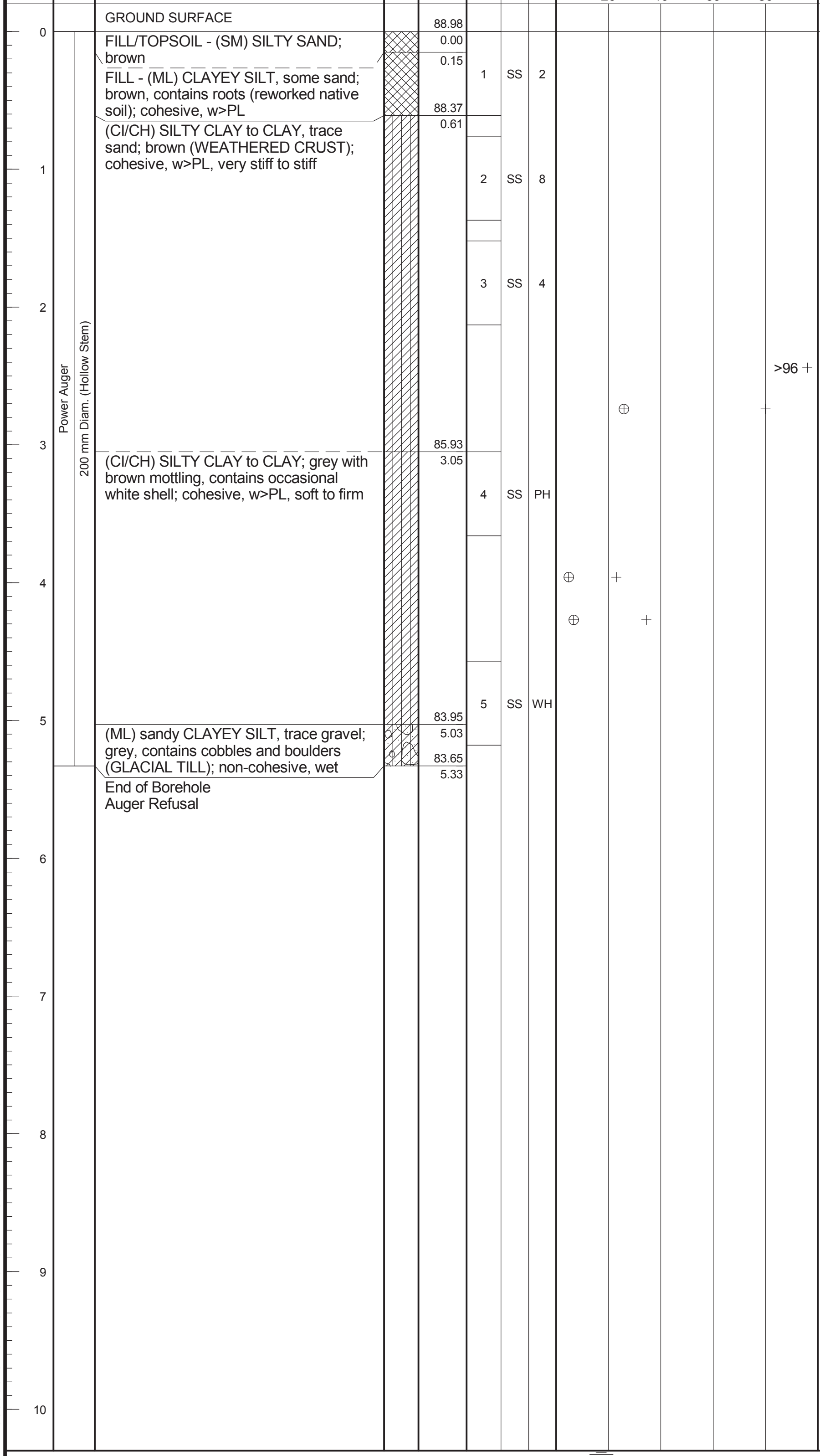
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl : 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: DWM

CHECKED: WAM

PROJECT: 1660030

RECORD OF BOREHOLE: 16-4

SHEET 1 OF 1

LOCATION: N 5034496.5 ;E 381166.8

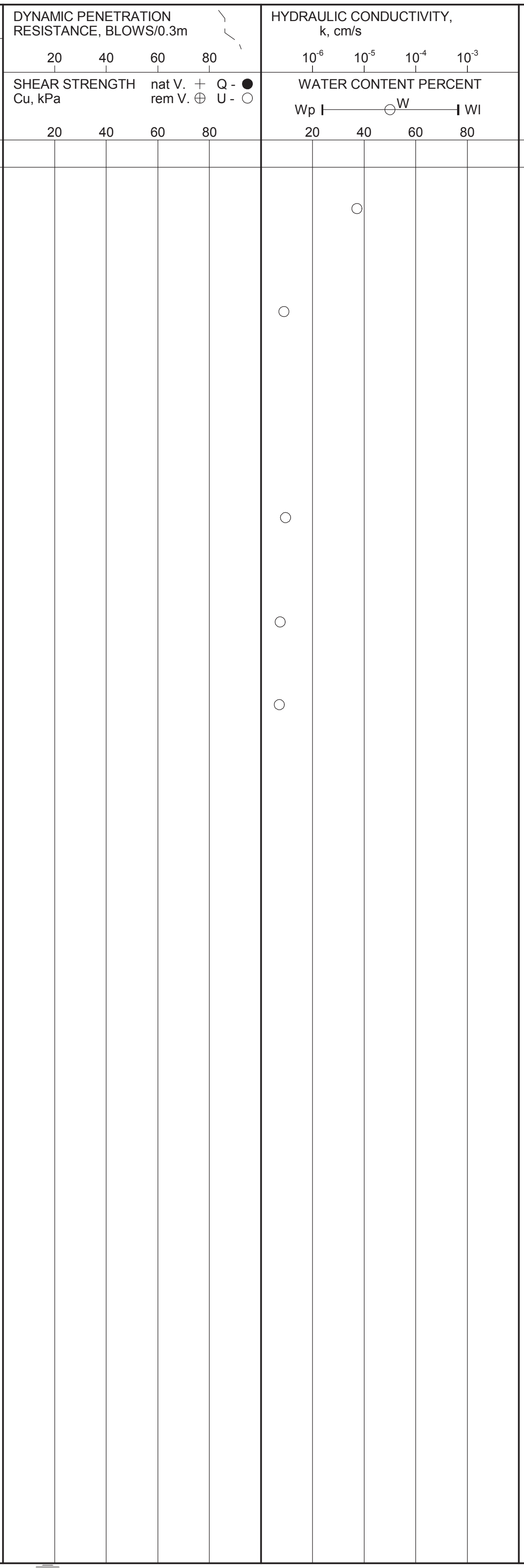
BORING DATE: November 1, 2016

DATUM: CGVD28

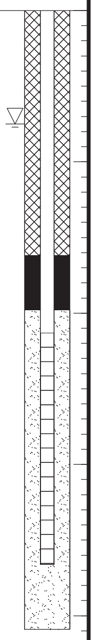
SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	90.14				
0	Power Auger 200 mm Diam. (Hollow Stem)	FILL/TOPSOIL - (ML) sandy SILT; brown	0.00				
		FILL - (SM) SILTY SAND; brown (reworked native soil); non-cohesive, moist, very loose	0.12	1	SS	3	
1		(SM/ML) SILTY SAND to sandy SILT, some gravel to gravelly; brown, contains cobbles and boulders (GLACIAL TILL); non-cohesive, moist, loose to very dense	89.23 0.91	2	SS	32	Cuttings
2				3	SS	20	Bentonite Seal Silica Sand
				4	SS	7	19 mm PVC Slot Screen
3		- grey at 3.0 m depth		5	SS	17	
4				6	SS	>50	
		End of Borehole Auger Refusal	86.05 4.09				W.L. in Screen at Elev. 89.39 m on Nov. 23, 2016



ADDITIONAL LAB. TESTING



DEPTH SCALE

1 : 50

LOGGED: DWM

CHECKED: WAM

PROJECT: 1660030

RECORD OF BOREHOLE: 16-5

SHEET 1 OF 1

LOCATION: N 5034535.9 ;E 381317.6

BORING DATE: November 10, 2016

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m				
		GROUND SURFACE		89.36							
0	Power Auger 200 mm Diam. (Hollow Stem)	FILL/TOPSOIL - (ML) sandy SILT; brown	▨	0.00							
		FILL - (ML) CLAYEY SILT, trace sand; brown with red mottling, contains rootlets (reworked native soil); cohesive, w<PL	▨	89.13 0.23	1	SS	5				
1		(CI/CH) SILTY CLAY to CLAY, trace sand; grey brown with red mottling (WEATHERED CRUST); cohesive, w>PL, very stiff	▨	88.75 0.61	2	SS	10				
2		(SM) SILTY SAND, some gravel; grey brown, contains cobbles and boulders (GLACIAL TILL); non-cohesive, moist, loose to very dense	▨	87.53 1.83	3	SS	6				
					4	SS	>50				
		End of Borehole Auger Refusal		86.82 2.54							
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: DWM

CHECKED: WAM

PROJECT: 1660030

RECORD OF BOREHOLE: 16-6

SHEET 1 OF 1

LOCATION: N 5034371.8 ;E 381143.3

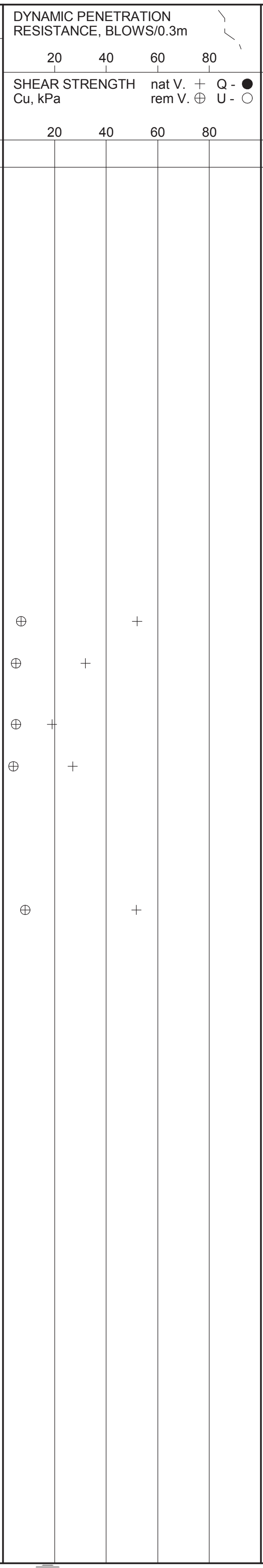
BORING DATE: November 1, 2016

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m
	GROUND SURFACE		88.72			
Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (ML) sandy SILT; brown		0.00			
	(CI/CH) SILTY CLAY to CLAY; grey brown with red mottling (WEATHERED CRUST); cohesive, w>PL, very stiff to stiff		88.52 0.20	1	SS	7
				2	SS	6
				3	SS	5
				4	SS	1
	(CI/CH) SILTY CLAY to CLAY; grey; cohesive, w>PL, soft to stiff		85.37 3.35			
				5	SS	PH
	End of Borehole Auger Refusal		83.19 5.53			



HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl; 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

DEPTH SCALE

1 : 50

LOGGED: DWM

CHECKED: WAM

PROJECT: 1660030

RECORD OF BOREHOLE: 16-7

SHEET 1 OF 1

LOCATION: N 5034408.7 ;E 381270.1

BORING DATE: November 2, 2016

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m
		GROUND SURFACE		88.71			
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (ML) sandy SILT; brown		0.00			
		(CI/CH) SILTY CLAY to CLAY, trace sand; grey brown with red mottling (WEATHERED CRUST); cohesive, w>PL, very stiff		0.08	1	SS	8
1					2	SS	5
2					3	SS	5
3		(CI/CH) SILTY CLAY to CLAY; grey; cohesive, w>PL, firm		85.66 3.05	4	SS	PH
5					5	SS	PH
6		(SM) gravelly SILTY SAND; grey, contains cobbles and boulders (GLACIAL TILL); non-cohesive, wet, loose to very dense		82.46 6.25	6	SS	5
					7	SS	>50
7		End of Borehole Auger Refusal		81.70 7.01			

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

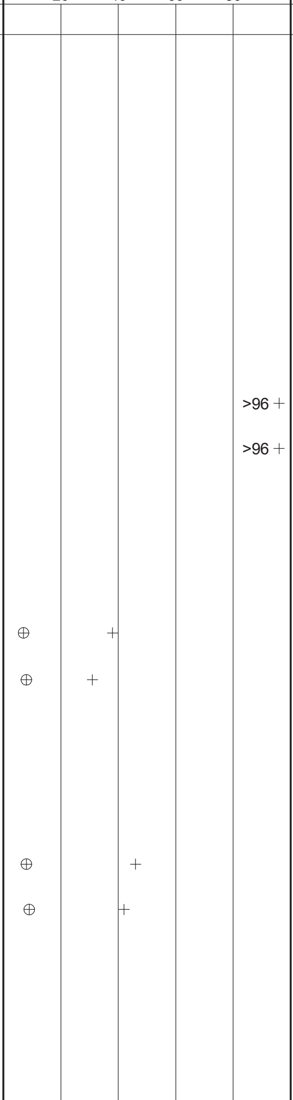
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ — 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl — 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



DEPTH SCALE

1 : 50

LOGGED: DWM

CHECKED: WAM

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4488-1 - TEST HOLE LOCATION PLAN

DRAWING PG4488-2 - BEDROCK CONTOUR PLAN

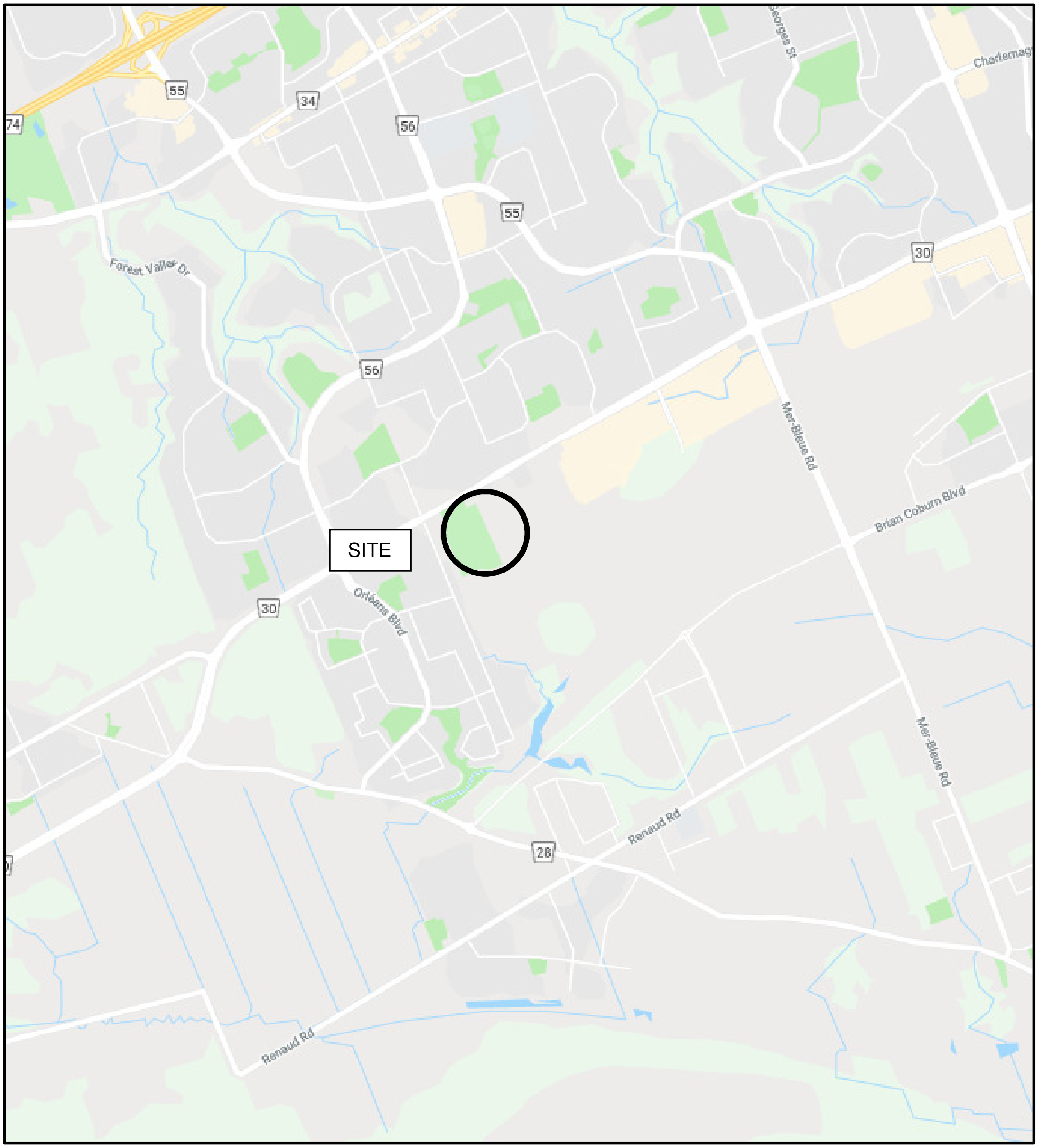


FIGURE 1

KEY PLAN

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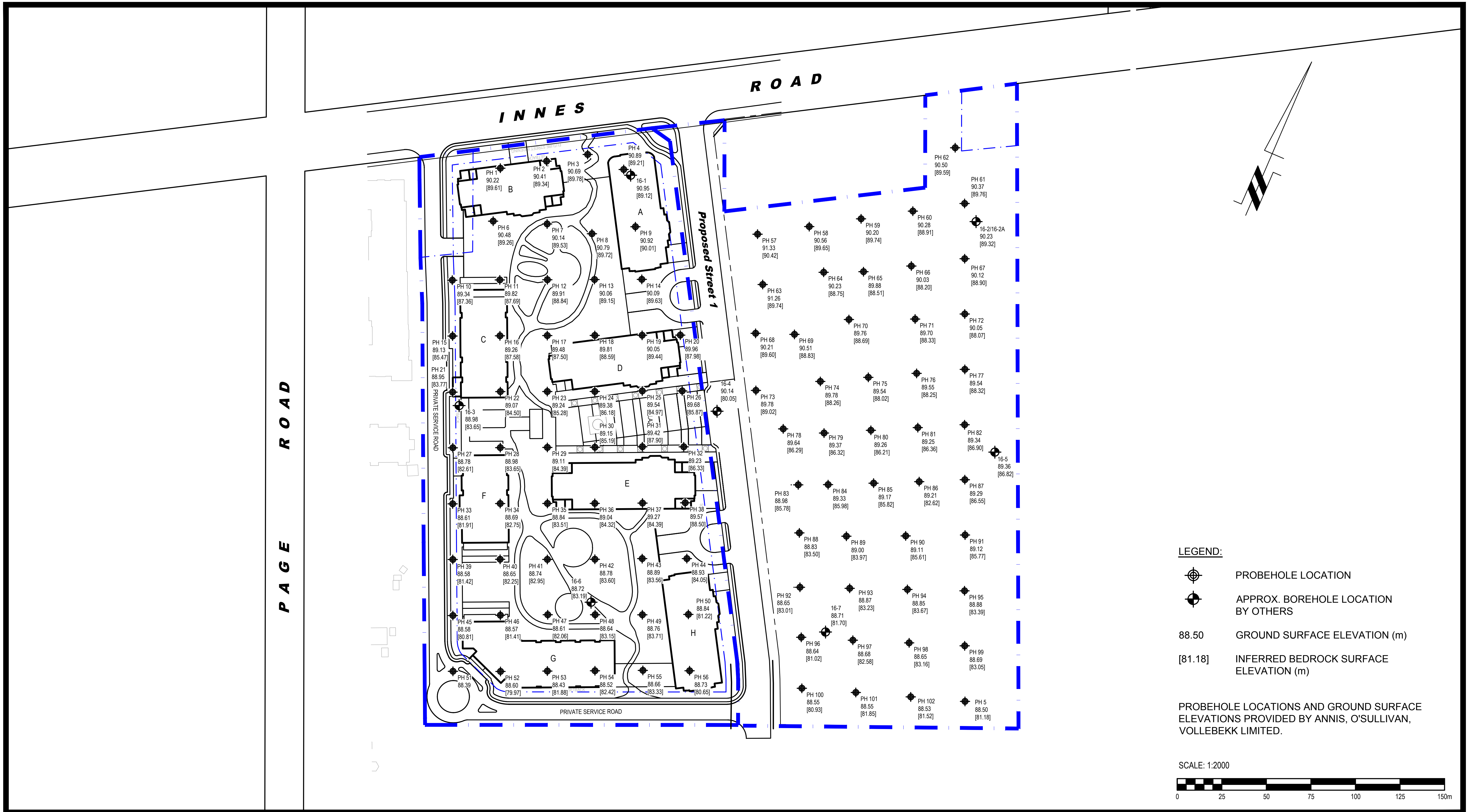

LEGEND:

⌖	PROBEHOLE LOCATION
⊕	APPROX. BOREHOLE LOCATION BY OTHERS
88.50	GROUND SURFACE ELEVATION (m)
[81.18]	INFERRED BEDROCK SURFACE ELEVATION (m)

PROBEHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LIMITED.

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NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO LATEST BASE PLAN	17/05/2019	FA

CANADIAN RENTAL DEVELOPMENT SERVICES

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY RESIDENTIAL BLDGS. - 3484, 3490 & 3592 INNES ROAD
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:2000
Drawn by: RCG
Checked by: FA
Approved by: DJG
Date: 07/2018
Report No.: PG4488-1
Dwg. No.: PG4488-1
Revision No.: 1

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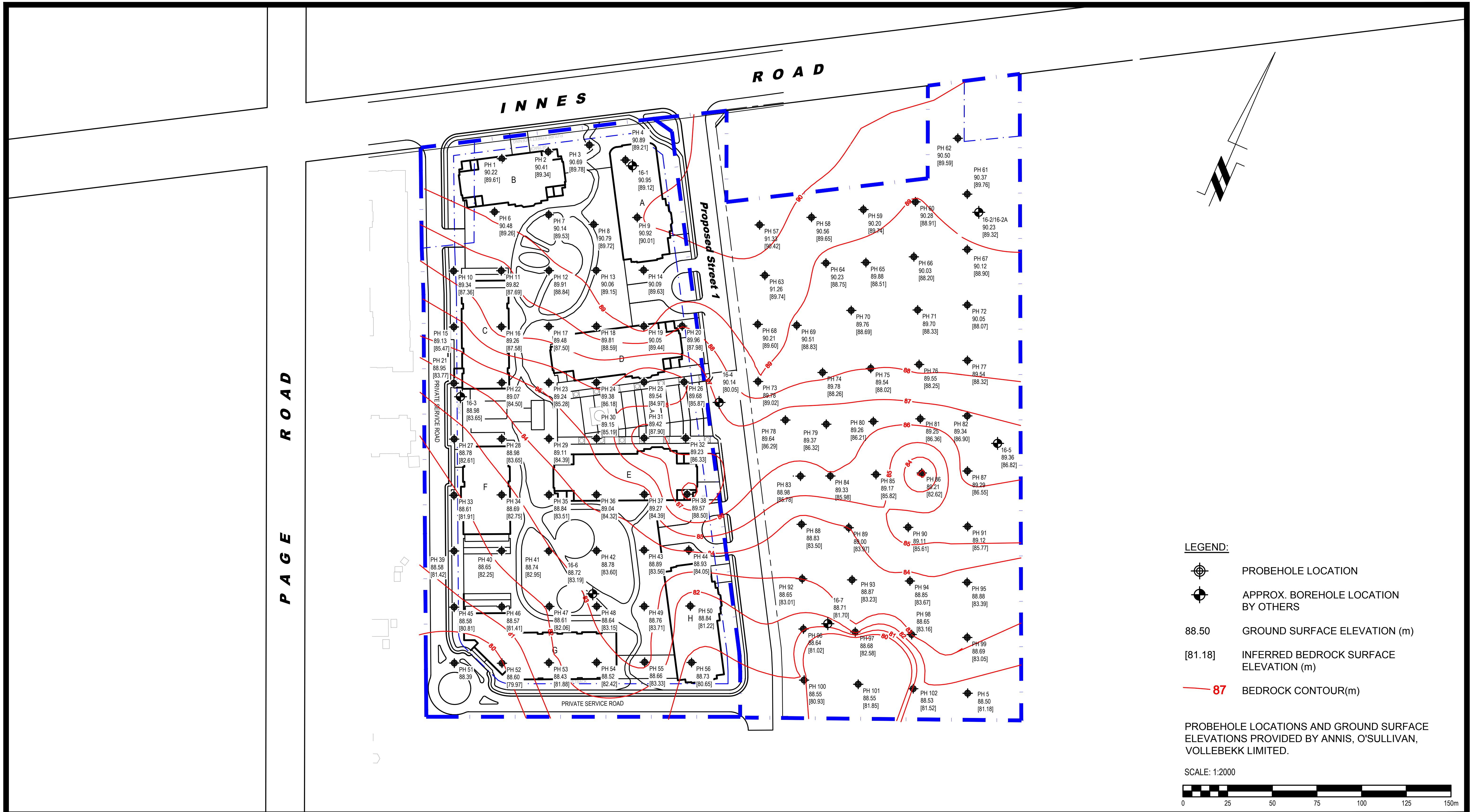

LEGEND:

⌖	PROBEHOLE LOCATION
⊕	APPROX. BOREHOLE LOCATION BY OTHERS
88.50	GROUND SURFACE ELEVATION (m)
[81.18]	INFERRED BEDROCK SURFACE ELEVATION (m)
87	BEDROCK CONTOUR(m)

PROBEHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LIMITED.

SCALE: 1:2000

0 25 50 75 100 125 150m

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consulting engineers

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NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO LATEST BASE PLAN	16/05/2019	FA

CANADIAN RENTAL DEVELOPMENT SERVICES
GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY RESIDENTIAL BLDGS. - 3484, 3490 & 3592 INNES ROAD
OTTAWA, ONTARIO

Title:
BEDROCK CONTOUR PLAN

Scale: 1:2000	Date: 09/2018
Drawn by: MPG	Report No.: PG4488-1
Checked by: SB	Dwg. No.: PG4488-2
Approved by: DJG	Revision No.: 1

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