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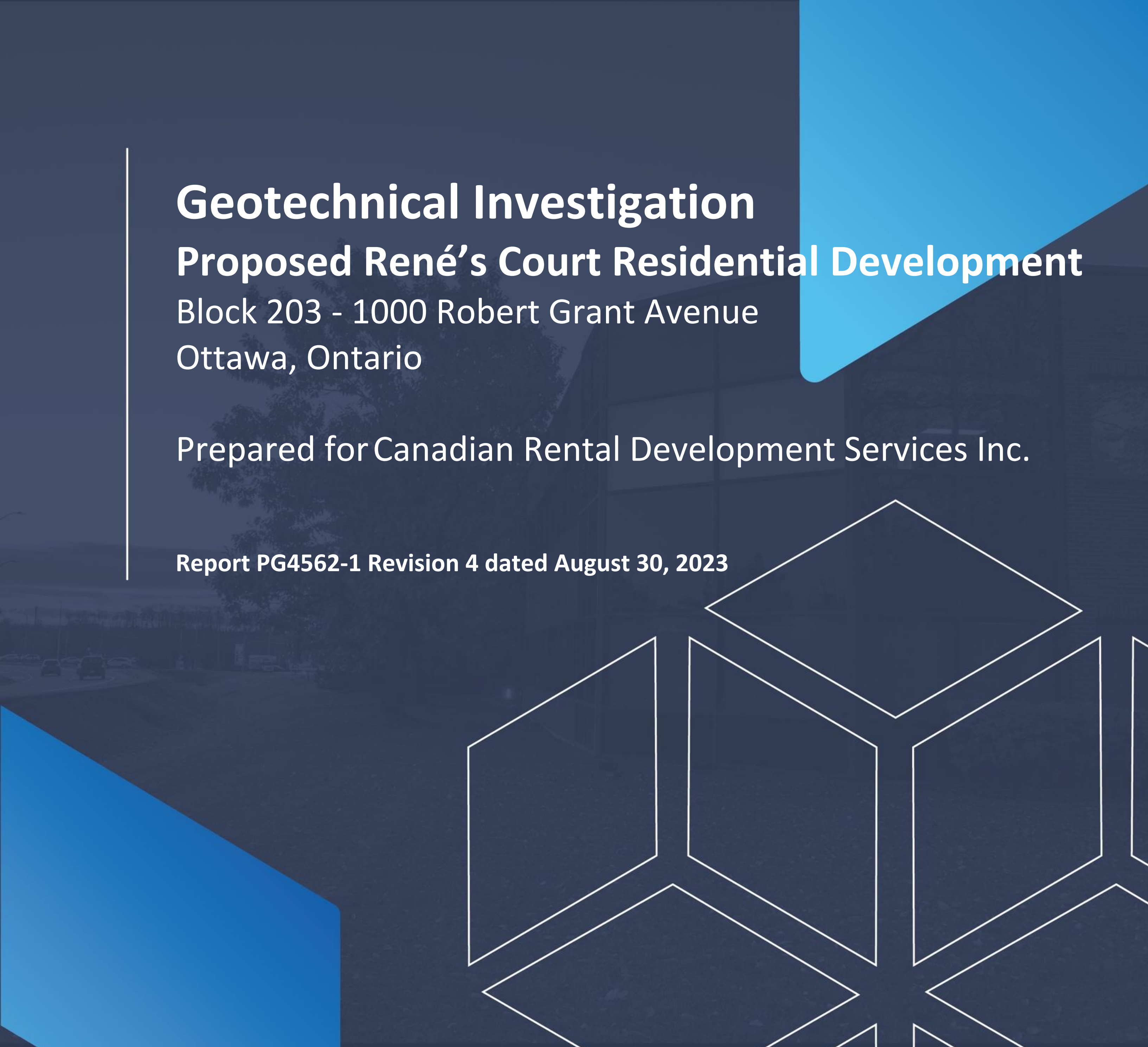

Geotechnical Investigation

Proposed René’s Court Residential Development

Block 203 - 1000 Robert Grant Avenue
Ottawa, Ontario

Prepared for Canadian Rental Development Services Inc.

Report PG4562-1 Revision 4 dated August 30, 2023

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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 René's Court Residential Development to be located on 1000 Robert Grant Avenue, in the City of Ottawa, Ontario (Refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the geotechnical investigation were to:

- ☐ Determine the subsurface soil and groundwater conditions based on the existing soils information and delineate the underlying bedrock across the subject site.

- ☐ Provide geotechnical recommendations pertaining to the design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Development

It is understood that the proposed development will consist of 3 multi-storey buildings as well as a 2-storey structure with 2 levels of shared underground parking structure occupying the majority of the subject site and a third underground parking level below the northwestern building along Robert Grant Avenue. Associated access lanes, hardscaped and landscaped areas are also anticipated for the subject development. It is further understood that the site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was completed on February 11, 2019. At that time, a total of 11 test pits were advanced to a maximum depth of 6.1 m below existing grade. A number of test pits were excavated within the immediate footprint and on the perimeter of the subject site to a maximum depth of 4 m below existing grade. A previous field program was carried out on August 15, 2018. At that time, 39 probe holes were advanced to a maximum depth of 10.1 m below existing grade to delineate the bedrock surface across the subject site. Existing geotechnical investigations were completed by others within the subject site and the adjacent sites in 2008. A supplemental geotechnical investigation program for bedrock delineation was completed on August 8, 2023, and consisted of advancing a total of 17 probe holes down to a maximum depth of 8.5 m below existing grade to delineate the bedrock surface across the subject site. The locations of the test holes are presented in the Drawing PG4562-1 - Test Hole Location Plan in Appendix 2.

The test holes were completed using a track mounted hydraulic shovel. The probe holes were advanced using a track mounted air-track drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department. The investigation procedure consisted of advancing each test hole to the bedrock surface in every location to delineate the bedrock surface across the site.

Sampling and In Situ Testing

Soil samples from the test pits were recovered from the side walls of the open excavation. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the grab samples were recovered from the test pits are shown as G on the Soil Profile and Test Data sheets in Appendix 1

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

3.2 Field Survey

The test hole locations were selected by Paterson taking into consideration existing site features. The historical test holes were surveyed and located in the field by Annis, O'Sullivan, Vollebekk Ltd. It is understood that the ground elevations at the test hole locations are referenced to a geodetic datum. The probeholes and ground surface elevation at each probehole location were surveyed by Paterson using a high precision GPS and referenced to a geodetic datum. The test hole locations and ground elevations at the test holes are presented on Drawing PG4562-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples recovered from the subject site were visually examined in our laboratory to review the field logs.

4.0 Observations

4.1 Surface Conditions

The majority of the subject site is currently undeveloped, and grass covered throughout. The ground surface across the subject site slopes down from south to north with a difference in elevation of approximately 6 m. It should be noted that fill piles were temporarily stored on site in 2015. The fill piles consisted of silty sand with gravel and trace clay. The fill piles were scattered across the central portion of the site with several locations where the original ground surface is exposed. Hydro lines were also observed to run east-west along the north property line.

The site is bordered by a residential development to the east, Robert Grant Avenue to the west and undeveloped lands to the south and north.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of topsoil and/or fill layer comprised of silty sand with gravel, occasional clay and organics. The above noted layers are underlain by a deposit of brown to grey silty clay within the south portion of the site. A silty clay deposit was encountered within the north portion of the site followed by a layer of glacial till. Bedrock was encountered in all test hole locations at depths ranging from ground surface within the south portion of the site and 10.1 m below existing grade along the north portion of the site. The depth of the bedrock encountered at each test hole location is presented on Drawing PG4562-1 - Test Hole Location Plan and Drawing PG4562-2 – Bedrock Contour Plan in Appendix 2.

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

Bedrock

Based on available geological mapping, the bedrock consists of interbedded dolostone and limestone of the Gull River formation with an overburden drift thickness ranging from ground surface to 10 m depth.

4.3 Groundwater

Based on groundwater observations and geotechnical investigations completed by others within the subject site and surrounding sites, the long-term groundwater level can be expected at an approximate elevation of 99 m where a deep clay deposit is present.

It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered adequate for the proposed development. The proposed multi-story buildings can be founded by conventional style shallow foundations placed on undisturbed, stiff silty clay, glacial till or a clean, surface sounded bedrock bearing surface. Buildings placed within the north portion of the subject site may require an alternative foundation, such as a raft foundation, end bearing piles or conventional footings placed over near vertical, zero entry, concrete in-filled trenches extending to a clean, surface sounded bedrock surface due to the presence of a deep silty clay deposit.

It is understood that an underground parking garage will occupy the majority of the subject site. Therefore, to accommodate the different bearing surfaces/foundation options used for the proposed structures (buildings and underground parking structure), control joints should be incorporated in the design to address differential settlement.

Due to the presence of a silty clay layer within the northern portion of the site, a permissible grade raise restriction will be required where footings will be founded on clay in the northern portion of the site. A permissible grade raise restriction of **2 m** is recommended for the north portion of the site.

Where 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 further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

Due to the proposed buildings extending below the bedrock surface throughout the south and northeast portions of the subject, it is expected all overburden material throughout those areas will be excavated from within the proposed building footprints.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming, rock grinding and controlled blasting will be required to remove the bedrock for the underground parking levels for several buildings. 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 the 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 or claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed the below noted vibration limits during the blasting program to reduce the risks of damage to the existing surrounding 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 carried out using near vertical sidewalls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing of the overburden. The 1 m horizontal ledge setback can be eliminated with a shoring program which has drilled piles extending below the proposed founding elevation.

Vibration Considerations

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

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

Two parameters are used to determine the permissible vibrations, namely, 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). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Bedrock Excavation Face Reinforcement and Preparation

Bedrock excavation face reinforcement methods, such as the use of horizontal rock anchors and rock wedges/bolts in conjunction with shotcrete and/or chain link fencing with a layer of woven geotextile connected to the excavation face is expected to be required at specific locations to prevent bedrock pop-outs, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface. Further, shotcrete and/or other material may be required to in-fill areas where bedrock pop-outs occur due to the nature of bedrock removal throughout the excavation footprint and in advance of the placement of foundation waterproofing products.

The requirement for bedrock excavation face reinforcement should be evaluated by Paterson personnel during the excavation operations. As a preliminary recommendation, provisions should be carried for providing a minimum 1 m wide bedrock face protection layer across building excavation footprint perimeters for all portions of the excavations that will extend below the bedrock surface. Throughout the building excavation and bedrock removal process, the vertical bedrock excavation perimeter surfaces should be hoe-rammed and grinded smooth to provide a relatively flat substrate surface for the placement of the drainage board. All loose bedrock fragments should be removed by grinding operations.

It is recommended that Paterson review the bedrock excavation program at the time of construction.

Overbreak in Bedrock

Sedimentary bedrock formation, such as limestone, dolomite and shale, contain bedding planes, joints and fractures, and mud seams which create natural planes of weakness within the rock mass. Although several factors of a blast may be controlled to reduce backbreak and overbreak, upon blasting, the rock mass will tend to break along natural planes of weakness that may be present beyond the designed blast profile. However, estimating the exact amount of backbreak and overbreak that may occur is not possible with conventional construction drill and blast methods.

Backbreak should be expected to occur along the perimeter of the building excavation footprint with conventional drill and blast bedrock removal methods. Further, overbreak is expected to occur throughout the lowest lifts of blasting due to the variable bedding planes and planes of weakness in the in-situ bedrock. It is very difficult to mitigate significant overblasting given the constraints posed by footing geometry and spacing with respect to the zone of influence of blasts and the bedrocks in-situ characteristics.

Depending on the methodology undertaken by the contractor, efforts taken to minimize backbreak and overbreak may add significant time and costs to the excavation operations and is not guaranteed to completely eliminate the potential for backbreak and overbreak. Overbreak below footings should be in-filled with lean-concrete and approved by Paterson prior to placing concrete.

As such, volume estimates of bedrock to be removed may not be reflective of the actual volume of bedrock that may be required to be removed at the time of construction. This may result in additional materials, such as imported fill and concrete, to make up for additional rock loss. It is recommended that the blasting operations be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

Fill Placement

Fill placed for grading throughout the building footprint should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. Imported fill material should be tested and approved prior to delivery to the site. The fill should be placed in a maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. These materials should be spread in a maximum of 300 mm thick loose lifts and compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in maximum 300 mm thick loose lifts to at least 98% of the material's SPMDD. The placement of subgrade material should be reviewed at the time of placement by Paterson personnel. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. This material should be tested and approved by Paterson 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 paved areas should be compacted to at least 100% of its SPMDD.

Under winter conditions, if snow and ice is present within the blast rock fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson personnel should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized. Providing a heat source during winter construction may be recommended should compacted fill material is intended to be exposed for long periods of time.

Fill Placement

Engineered fill placed for grading beneath the building footprints, where required, should consist 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 a maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the buildings and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

5.3 Foundation Design

Bearing Resistance Values (Conventional Shallow Foundation)

Footings placed on a clean, surface sounded bedrock surface can be designed using a bearing resistance value at 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.

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 **250 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 bearing resistance value at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen, or disturbed soil, have been removed, in dry weather conditions, 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.

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

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 sub excavation 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.

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. A clean, surface sounded bedrock bearing medium will require a lateral support zone of 1H:6V (or flatter). Also, a weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

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 (**20 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 300 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 **2,000 kPa**.

Raft Foundation

Alternatively, for the north portion of the site, consideration can be given to a raft foundation if the building loads exceed the bearing resistance values provided for a conventional shallow footing foundation. The following parameters may be used for raft design. For design purposes, it was assumed that the base of the raft foundation will be located at a 7 m depth with two underground levels.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The bearing resistance value at SLS (contact pressure) of **170 kPa** will be considered acceptable. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The contact pressure provided considers the stress relief associated with the soil removal required for proposed buildings. The factored bearing resistance (contact pressure) at ULS can be taken as **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **4 MPa/m** for a contact pressure of **170 kPa**. The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Based on the following assumptions for the raft foundation, the proposed buildings can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

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 applied, provided a site-specific shear wave velocity test is 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 Floor 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 building(s) 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 structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 20 kN/m^3.

However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, 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 = $(1.45\text{-}a_{max}/g)a_{max}$
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 site area is 0.32 g 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 $P_o = 0.5\ K_o\ \gamma\ H^2$, 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 - At-Grade Parking

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

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

Table 3 - Recommended Flexible Pavement Structure - Above Podium

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
SUBGRADE - Concrete Podium - A waterproofing membrane should be applied directly above the podium concrete followed by a protection board. A 50 mm thick layer of rigid insulation such as HI-60 or equivalent should be placed above the protection board.	

Table 4 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
150	**32 MPa Concrete**
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed bedrock.	

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 I or 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.

Pavement Structure Drainage

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

Due to the impervious nature of the subgrade materials consideration should be given to installing subdrains at each catch basin during the pavement construction. These drains should be at least 3 m long and extend in four orthogonal directions or longitudinally when placed along a curb. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

The following recommendations may be considered for the architectural design of the buildings foundation drainage systems. It is recommended that Paterson be engaged at the design stage of the future buildings (and prior to tender) to review and provide supplemental information for the buildings foundation drainage system design.

Supplemental details, review of architectural design drawings and additional information may be provided by Paterson for these items for incorporation in the building design packages and associated tender documents. It is recommended that Paterson review all details associated with the foundation drainage system prior to tender.

Groundwater Suppression System

It is recommended that a groundwater suppression system be provided for the proposed structures. It is expected that the foundation wall will be cast as a blind-sided pour against a shoring system and the bedrock surface. It is recommended that the groundwater suppression system consist of the following:

☐ A waterproofing membrane should be placed against the shoring system between underside of footings and 2 m below existing ground surface. The height of the waterproofing layer should be confirmed on a per-building basis, however, is expected to vary between 2 and 3 m below existing ground surface. Where the membrane will extend below the bedrock surface, it is recommended to consist of a membrane with a bentonite-lined face for being paced against the bedrock surface. The membrane is recommended to overlap below the overlying perimeter foundation footprint by a minimum of 1 m inwards towards the building footprint and from the face of the overlying foundation. This will allow construction to proceed without imposing groundwater lowering within the surrounding area of the proposed buildings in the short and long term conditions.

☐ A composite drainage membrane (DeltaDrain 6000, MiraDrain G100N or equivalent) should be placed against the HDPE face of the waterproofing membrane with the geotextile layer facing the waterproofing layer from finished ground surface to the top of the footing.

- ☐ The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by the geotechnical consultant. It is highly recommended that the drainage board rolls be installed horizontally rather than vertically to minimize the number of vertical joints forming between the rolls.

- ☐ The bedrock face, where located within a buildings excavation, is recommended to be grinded to provide a smooth-surface for the installation of the waterproofing layer. Large cavities should be reviewed by Paterson as the excavation progresses to assess the requirement to in-fill cavities suitably to facilitate the installation of the waterproofing layer.

- ☐ It is recommended that 150 mm diameter PVC sleeves at 6 m centers be cast in the foundation wall at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The sleeves should be connected to openings in the HDPE face of the drainage board layer. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area via an underfloor and interior drainage pipe system.

The top of the foundation drainage board should be provided with a suitable termination bar against the foundation wall to mitigate the potential for water to perch between the drainage board and foundation wall.

Interior Perimeter and Underfloor Drainage

The interior perimeter and underfloor drainage system will be required to control water infiltration below the lowest underground parking level slab and redirect water from the building's foundation drainage system to the buildings sump pit(s). The interior perimeter and underfloor drainage pipe should consist of a 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided with tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves. The spacing of the underfloor drainage system should be confirmed by Paterson once the foundation layout and sump system location has been finalized.

Elevator Pit Waterproofing

The elevator shaft exterior foundation walls should be waterproofed to avoid any infiltration into the elevator pit. It is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved other) be applied to the exterior of the elavator shaft foundation wall.

The Colphene Torch'n Stick waterproofing membrane should extend over the vertical portion of the raft slab and down to the top of the footing in accordance with the manufacturer's specifications. A continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the interface between the concrete base slab below the elevator shaft foundation walls.

The 150 mm diameter perforated corrugated pipe underfloor drainage should be placed along the perimeter of the exterior sidewalls and provided a gravity connection to the sump pump basin or the elevator sump pit.

Foundation Backfill

Above the bedrock surface, 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 Miradrain G100N or 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.

Podium Deck Waterproofing Tie-In

Waterproofing layers for podium deck surfaces should overlap across and below the top end lap of the vertically installed composite foundation drainage board to mitigate the potential for water to migrate between the drainage board and foundation wall and as depicted in Figure 3 – Podium Deck to Foundation Wall Drainage System Tie-In Detail.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

Foundation Raft Slab Construction Joints

Where a raft slab is being considered, it is anticipated the raft slab will be poured in several pour segments. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab.

Finalized Drainage and Waterproofing Design

Paterson should be provided with the finalized structural and architectural drawings for each building to provide a building specific waterproofing and drainage design which includes the above noted recommendations. The design will provide recommendations for other items such as minimum pipe spacings, pipe mechanical connections below grade, transitioning from blind to double sided pours (if applicable), etc.

6.2 Protection Against Frost Action

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

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 600 mm m of soil cover, in conjunction with foundation insulation and as reviewed and advised by Paterson during the design stage, 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 temporary shoring system may be calculated with the following parameters.

Table 5 – Soils Parameter for Shoring System Design

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.

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. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

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. The locations of the proposed clay seals are shown on Figure 2 - Proposed Clay Seal Locations.

6.5 Groundwater Control

Groundwater Control for Building Construction

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

6.6 Winter Construction

The subsurface conditions at this site consist of a combination of frost susceptible materials (fill material and silty clay) and non-frost susceptible material (bedrock). In presence of water and freezing conditions ice could form within the frost susceptible 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, it is expected that the founding medium will mainly consist of clean, surface sounded bedrock surface. Therefore, no frost protection measures are expected to be required for the founding medium provided that bearing medium inspections are completed by Paterson at the time of excavation. However, where the founding medium consists of frost susceptible material, 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.

6.7 Landscaping Considerations

Tree Planting Restrictions

According to the City of Ottawa Guidelines for tree planting, where a sensitive silty clay deposit is present within the vicinity of the site, tree planting restrictions should be determined. However, for this site, 2 or 3 levels of underground parking are proposed to occupy the majority of the site. Therefore, based on the encountered subsurface conditions and the founding level of the proposed buildings, tree planting restrictions are not required from a geotechnical perspective.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once and future details of the proposed development have been prepared:

- ☐ Review preliminary and detailed grading, servicing, landscaping and structural plan(s) from a geotechnical perspective.
- ☐ Review of the geotechnical aspects of the excavation contractor's shoring design, if not design by Paterson, prior to construction, if applicable.
- ☐ Review of architectural plans pertaining to groundwater suppression system, underfloor drainage systems and waterproofing details for elevator shafts.

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

- ☐ Review and inspection of the installation of the foundation drainage systems.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Observation of driving and re-striking of all pile foundations.
- ☐ 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 and follow-up field density tests to determine the level of compaction achieved.
- ☐ 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 the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

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 agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Zubaida Al-Moselly, P.Eng

Maha Saleh, M.A.Sc., P.Eng.

LICENSED PROFESSIONAL ENGINEER
August 30, 2023
M. SALEH
100507739
PROVINCE OF ONTARIO

Report Distribution:

❑ Canadian Rental Development Services Inc. (email copy)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

TEST HOLE LOGS BY OTHERS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1000 Robert Grant Avenue
Ottawa, Ontario

DATUM Geodetic elevations provided by Annis O'Sullivan Vollebekk Ltd.

REMARKS

BORINGS BY Excavator

DATE February 11, 2019

FILE NO. PG4562

HOLE NO. TP 1-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
Ground Surface						0	103.91		
Brown **SILTY CLAY**		G	1			1	102.91		
		G	2			2	101.91		
		G	3			3	100.91		
-grey by 3.5m depth									
		G	4			4	99.91		
						5	98.91		
5.40									
GLACIAL TILL: Grey silty sand with clay		G	5						
-with cobbles at 5.5m depth 5.80		G	6						
End of Test Pit Refusal to excavation on bedrock surface @ 5.8m depth									

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1000 Robert Grant Avenue
Ottawa, Ontario

DATUM Geodetic elevations provided by Annis O'Sullivan Vollebekk Ltd.

FILE NO. PG4562

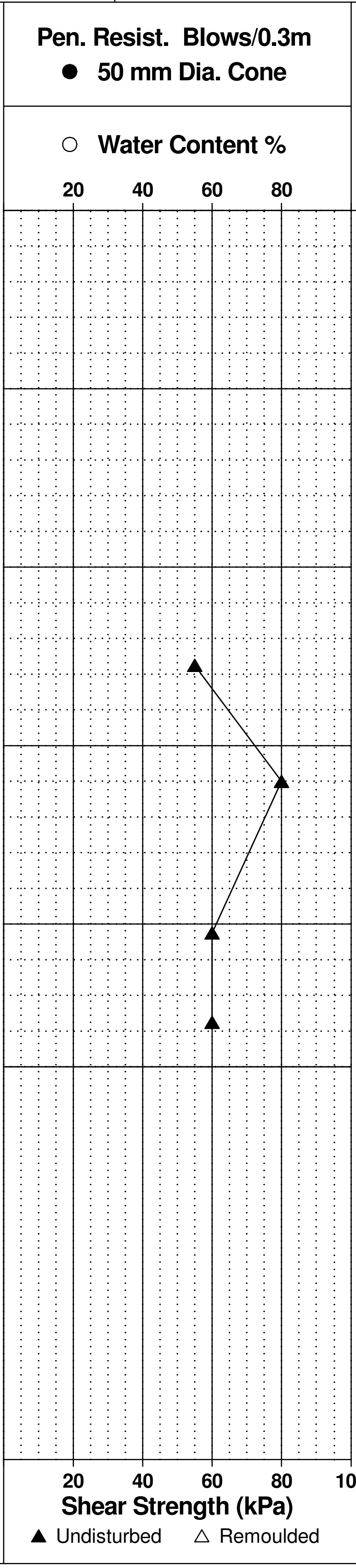
REMARKS

HOLE NO. TP 2-19

BORINGS BY Excavator

DATE February 11, 2019

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
Ground Surface						0	101.85	
TOPSOIL 0.20								
		G	1			1	100.85	
Stiff, brown **SILTY CLAY**								
		G	2			2	99.85	
		G	3					
-grey by 2.6m depth						3	98.85	
3.50		G	4					
Grey **SILTY CLAY** with sand		G	5			4	97.85	
4.80		G	6					
End of Test Pit Refusal to excavation on bedrock surface @ 4.8m depth								



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SOIL PROFILE AND TEST DATA

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DATUM Geodetic elevations provided by Annis O'Sullivan Vollebekk Ltd.

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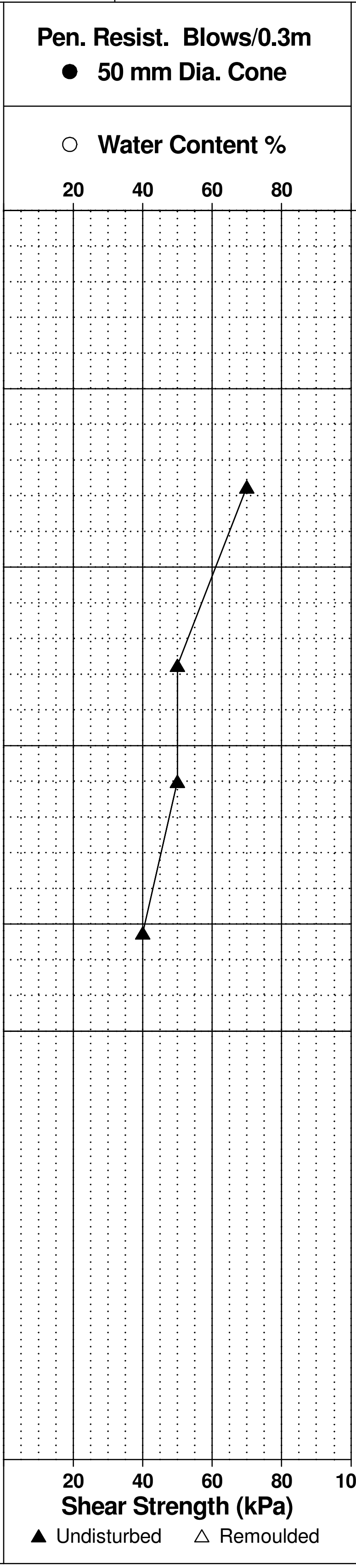
DATE February 11, 2019

FILE NO. PG4562

HOLE NO. TP 3-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
Ground Surface						0	102.16	
TOPSOIL 0.30	■							
Stiff, brown **SILTY CLAY**	▨	G	1			1	101.16	
	▨	G	2					
2.30	▨					2	100.16	
Stiff to firm, grey **SILTY CLAY** with sand, trace gravel	▨	G	3					
	▨					3	99.16	
	▨	G	4					
	▨	G	5			4	98.16	
4.60								
End of Test Pit Refusal to excavation on bedrock surface @ 4.6m depth								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1000 Robert Grant Avenue
Ottawa, Ontario

DATUM Geodetic elevations provided by Annis O'Sullivan Vollebekk Ltd.

REMARKS

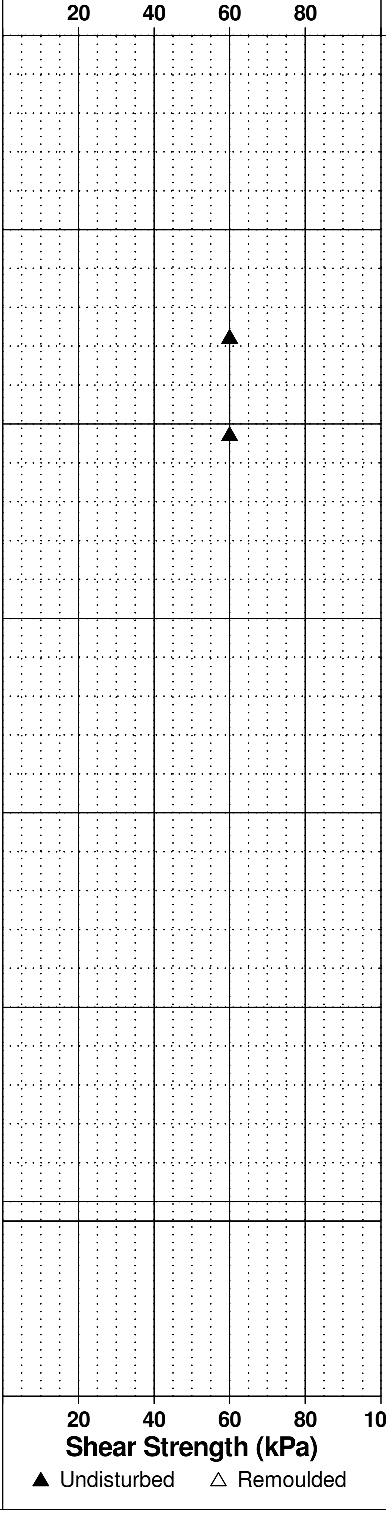
BORINGS BY Excavator

DATE February 11, 2019

FILE NO. PG4562

HOLE NO. TP 4-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	103.67
TOPSOIL	■						
0.45							
Stiff, brown **SILTY CLAY**	▨	G	1			1	102.67
	▨	G	2				
-grey by 1.8m depth	▨	G	3			2	101.67
	▨					3	100.67
	▨	G	4				
	▨	G	5			4	99.67
	▨	G	6				
4.90							
Grey **SILTY CLAY** with sand	▨	G	7			5	98.67
	▨	G	8				
6.10						6	97.67
End of Test Pit							
Refusal to excavation on bedrock surface @ 6.1m depth							



Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1000 Robert Grant Avenue
Ottawa, Ontario

DATUM Geodetic elevations provided by Annis O'Sullivan Vollebekk Ltd.

REMARKS

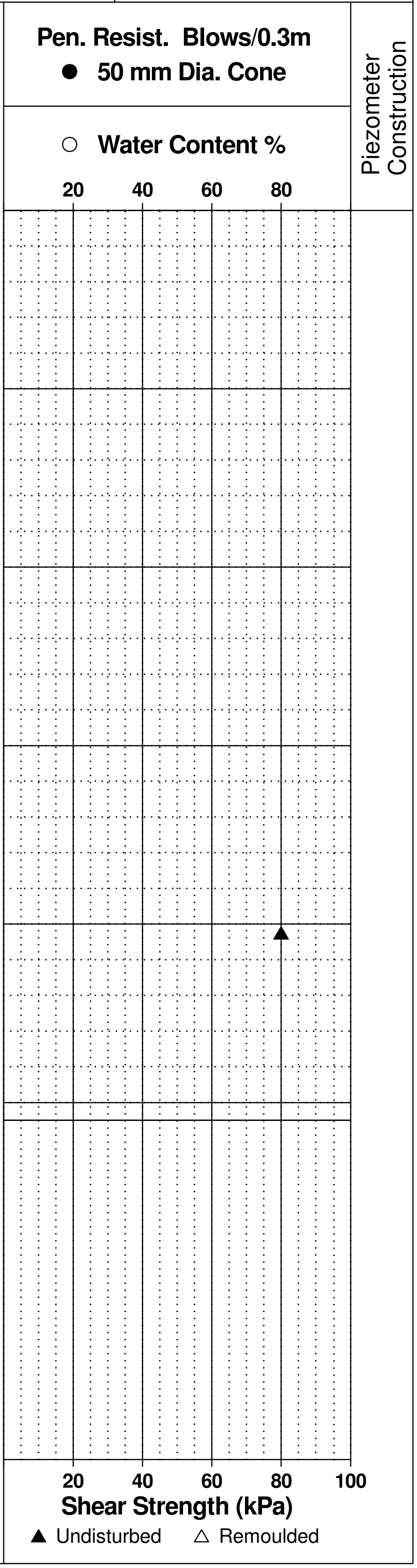
BORINGS BY Excavator

DATE February 11, 2019

FILE NO. PG4562

HOLE NO. TP 5-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	104.08
TOPSOIL 0.30	■						
Brown **SILTY CLAY**	▨	G	1			1	103.08
	▨	G	2				
-grey by 2.0m depth	▨	G	3			2	102.08
	▨	G	4				
	▨	G	5			3	101.08
4.30	▨	G	6			4	100.08
Grey **SILTY CLAY** with sand 5.10	▨	G	7			5	99.08
End of Test Pit Refusal to excavation on bedrock surface @ 5.1m depth							



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1000 Robert Grant Avenue
Ottawa, Ontario

DATUM Geodetic elevations provided by Annis O'Sullivan Vollebekk Ltd.

REMARKS

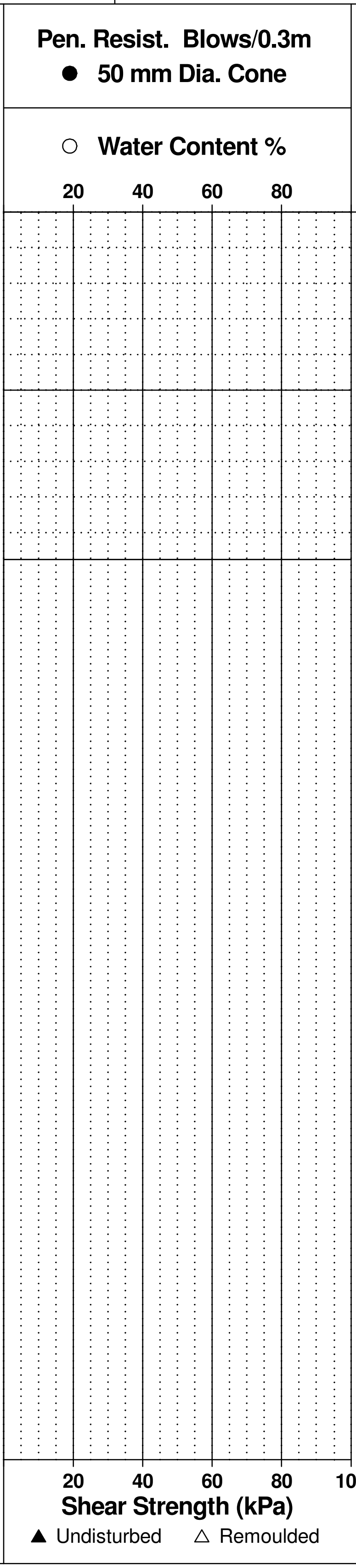
BORINGS BY Excavator

DATE February 11, 2019

FILE NO. PG4562

HOLE NO. TP 6-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			
Ground Surface						0	103.46	
TOPSOIL 0.45	■							
Brown **SILTY CLAY**	▨	G	1			1	102.46	
-grey by 1.5m depth 1.95	▨	G	2					
End of Test Pit Refusal to excavation on bedrock surface @ 1.95m depth								



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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DATUM Geodetic elevations provided by Annis O'Sullivan Vollebekk Ltd.

REMARKS

BORINGS BY Excavator

DATE February 11, 2019

FILE NO. PG4562

HOLE NO. TP 7-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
Ground Surface						0	104.22		
TOPSOIL 0.08									
Brown **SILTY CLAY** 1.00		G	1			1	103.22		
End of Test Pit Refusal to excavation on bedrock surface @ 1.0m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1000 Robert Grant Avenue
Ottawa, Ontario

DATUM Geodetic elevations provided by Annis O'Sullivan Vollebekk Ltd.

REMARKS

BORINGS BY Excavator

DATE February 11, 2019

FILE NO. PG4562

HOLE NO. TP 8-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface							
TOPSOIL 0.08	■					0	104.09
Brown **SILTY CLAY**	▨	G	1				
	▨	G	2			1	103.09
1.80							
End of Test Pit Refusal to excavation on bedrock surface @ 1.8m depth							

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

○ **Water Content %**

20 40 60 80

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

Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1000 Robert Grant Avenue
Ottawa, Ontario

DATUM Geodetic elevations provided by Annis O'Sullivan Vollebekk Ltd.

REMARKS

BORINGS BY Excavator

DATE February 11, 2019

FILE NO. PG4562

HOLE NO. TP 9-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
Ground Surface									
TOPSOIL 0.30	■					0	104.71		
	▨	G	1						
	▨	G	2						
Brown **SILTY CLAY**	▨					1	103.71		
2.20	▨					2	102.71		
End of Test Pit Refusal to excavation on bedrock surface @ 2.2m depth									

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

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SOIL PROFILE AND TEST DATA

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REMARKS

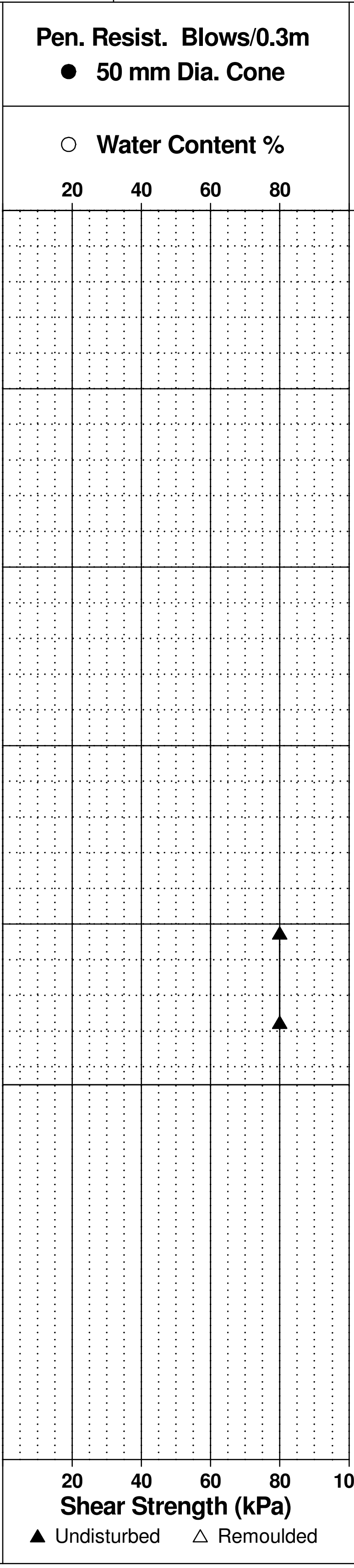
BORINGS BY Excavator

DATE February 11, 2019

FILE NO. PG4562

HOLE NO. TP10-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
Ground Surface						0	104.12
FILL: Gravelly sand with clay and cobbles						1	103.12
						2	102.12
2.20							
TOPSOIL							
2.50							
Very stiff, brown/grey **SILTY CLAY**		G	1			3	101.12
		G	2				
-stiff and grey by 4.0m depth		G	3			4	100.12
		G	4				
4.90							
End of Test Pit Refusal to excavation on bedrock surface @ 4.9m depth							



Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1000 Robert Grant Avenue
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REMARKS

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DATE February 11, 2019

FILE NO. **PG4562**

HOLE NO. **TP11-19**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
Ground Surface						0	103.56		
FILL: gravelly sand with clay and cobbles	▨					1	102.56		
1.50									
TOPSOIL	■								
1.80									
Brown **SILTY CLAY**	▨	G	1			2	101.56		
-grey by 2.4m depth		G	2						
		G	3			3	100.56		
		G	4						
		G	5			4	99.56		
4.40									
End of Test Pit Refusal to excavation on bedrock surface @ 4.4m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

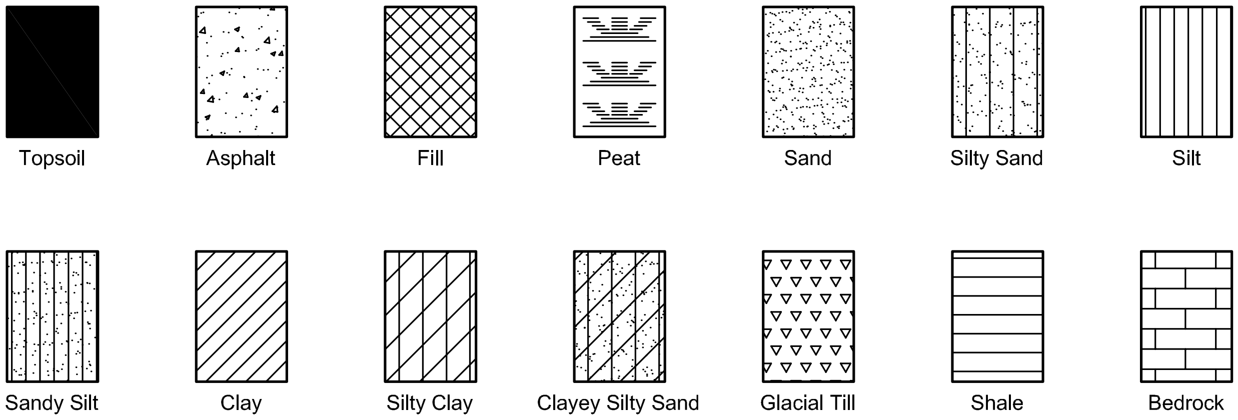
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

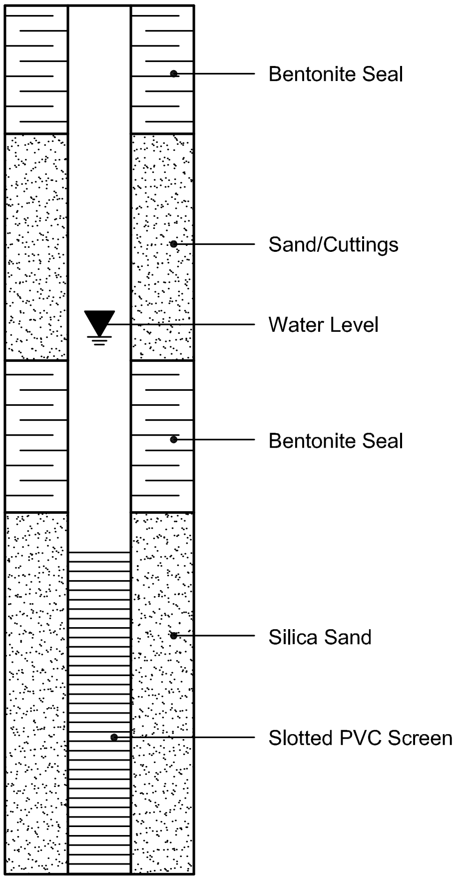
SYMBOLS AND TERMS (continued)

STRATA PLOT

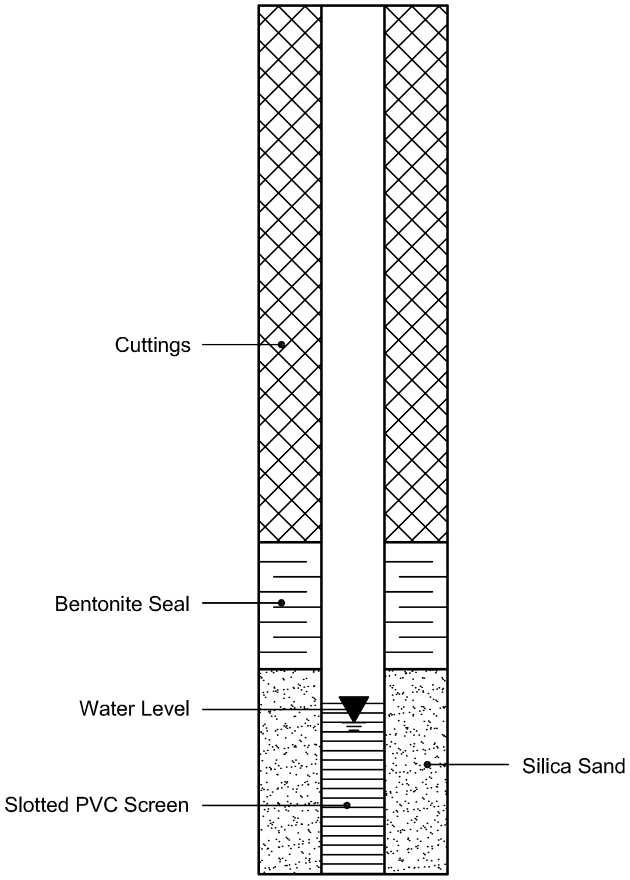


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PROJECT: 08-601

RECORD OF TEST PIT 08-14

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Not applicable

DATE OF EXCAVATION: November 6 and 7, 2008

TYPE OF EXCAVATOR: CAT 320D

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ 20 40 60 80	WATER CONTENT (PERCENT) Wp W Wl 20 40 60 80	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
	Ground Surface							
0	TOPSOIL							
	Brown SILTY SAND		0.23					
	Practical shovel refusal on BEDROCK End of test pit		0.51					No groundwater inflow observed on completion of excavating.
1								
2								
3								
4								
5								

DEPTH SCALE

1 to 25

Houle Chevrier Engineering Ltd.

LOGGED: AN

CHECKED:

TESTPIT RECORD 2011 GINT LOGS NOVEMBER 10 2008.GPJ HCE DATA TEMPLATE.GDT 3/10/11

PROJECT: 08-601

RECORD OF TEST PIT 08-15

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Not applicable

DATE OF EXCAVATION: November 6 and 7, 2008

TYPE OF EXCAVATOR: CAT 320D

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ 20 40 60 80	WATER CONTENT (PERCENT) Wp ├──○ W──┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface TOPSOIL							
	Very stiff to stiff grey brown SILTY CLAY, trace sand (weathered crust)	▨	0.25					
1				1				
	Practical shovel refusal on BEDROCK End of test pit		1.63					▽ Groundwater inflow at 1.63 metres below ground surface on completion of excavation
2								
3								
4								
5								

DEPTH SCALE

1 to 25

Houle Chevrier Engineering Ltd.

LOGGED: AN

CHECKED:

PROJECT: 08-601

RECORD OF TEST PIT 08-26

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Not applicable

DATE OF EXCAVATION: November 6 and 7, 2008

TYPE OF EXCAVATOR: CAT 320D

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ 20 40 60 80	WATER CONTENT (PERCENT) Wp ⊢ W ○ ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
	Ground Surface							
0	TOPSOIL							
	Fractured BEDROCK		0.11					
	Practical shovel refusal on BEDROCK End of test pit		0.31					No groundwater inflow observed on completion of excavating.
1								
2								
3								
4								
5								

DEPTH SCALE

1 to 25

Houle Chevrier Engineering Ltd.

LOGGED: AN

CHECKED:

PROJECT: 08-601

RECORD OF TEST PIT 08-27

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Not applicable

DATE OF EXCAVATION: November 6 and 7, 2008

TYPE OF EXCAVATOR: CAT 320D

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ (20, 40, 60, 80)	WATER CONTENT (PERCENT) Wp ⊢—○ W—⊣ Wl (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface TOPSOIL						
	Very stiff to stiff grey brown SILTY CLAY (weathered crust) transitioning to grey SILTY CLAY at depth	0.23					
1							
2							
3	Grey sandy silt, some gravel, cobbles, and boulders (GLACIAL TILL)	2.87	1				
	Practical shovel refusal on BEDROCK End of test pit	3.20					No groundwater inflow observed on completion of excavating.
4							
5							

DEPTH SCALE

1 to 25

Houle Chevrier Engineering Ltd.

LOGGED: AN

CHECKED:

PROJECT: 08-601

RECORD OF TEST PIT 08-37

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Not applicable

DATE OF EXCAVATION: November 6 and 7, 2008

TYPE OF EXCAVATOR: CAT 320D

DEPTH SCALE METRES	SOIL PROFILE			SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ 20 40 60 80	WATER CONTENT (PERCENT) Wp ⊢—○ W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)					
	Ground Surface							
0	TOPSOIL							
	Very stiff to stiff grey brown SILTY CLAY (weathered crust)		0.25					
1								
2								∇
3								
	Grey SILTY CLAY		3.35					
4	End of test pit		3.96					Groundwater inflow at 2.26 metres below ground surface on completion of excavation.
5								

DEPTH SCALE

1 to 25

Houle Chevrier Engineering Ltd.

LOGGED: AN

CHECKED:

PROJECT: 08-601

RECORD OF TEST PIT 08-41

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Not applicable

DATE OF EXCAVATION: November 6 and 7, 2008

TYPE OF EXCAVATOR: CAT 320D

DEPTH SCALE METRES	SOIL PROFILE			SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ 20 40 60 80	WATER CONTENT (PERCENT) Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)					
	Ground Surface							
0	TOPSOIL							
	Fractured BEDROCK		0.08					
	Practical shovel refusal on BEDROCK End of test pit		0.42					No groundwater inflow observed on completion of excavating.
1								
2								
3								
4								
5								

DEPTH SCALE

1 to 25

Houle Chevrier Engineering Ltd.

LOGGED: AN

CHECKED:

PROJECT: 08-601

RECORD OF TEST PIT 08-42

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Not applicable

DATE OF EXCAVATION: November 6 and 7, 2008

TYPE OF EXCAVATOR: CAT 320D

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕	WATER CONTENT (PERCENT) Wp W Wl	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface TOPSOIL						
	Very stiff to stiff grey brown SILTY CLAY (weathered crust)	0.28					
	Grey SILTY CLAY	2.44					▽
4	End of test pit	3.99					Groundwater inflow at 2.49 metres below ground surface on completion of excavation.

Depth scale marks: 0, 1, 2, 3, 4, 5. Shear strength scale: 20, 40, 60, 80. Water content scale: 20, 40, 60, 80.

DEPTH SCALE

1 to 25

Houle Chevrier Engineering Ltd.

LOGGED: AN

CHECKED:

PROJECT: 08-601

RECORD OF TEST PIT 08-47

SHEET 1 OF 1

LOCATION: See Site Plan, Figure 2

DATUM: Not applicable

DATE OF EXCAVATION: November 6 and 7, 2008

TYPE OF EXCAVATOR: CAT 320D

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ 20 40 60 80	WATER CONTENT (PERCENT) Wp ├──○ W──┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
	Ground Surface							
0	TOPSOIL							
1 2	Very stiff to stiff grey brown SILTY CLAY, transitioning to grey SILTY CLAY at depth	▨	0.23					▽
3 4 5	Practical shovel refusal on BEDROCK End of test pit		2.69					Groundwater inflow at 1.93 metres below ground surface on completion of excavation.

DEPTH SCALE

1 to 25

Houle Chevrier Engineering Ltd.

LOGGED: AN

CHECKED:

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - PROPOSED CLAY SEAL LOCATIONS

DRAWING PG4562-1 - TEST HOLE LOCATION PLAN

DRAWING PG4562-2 - BEDROCK CONTOUR PLAN

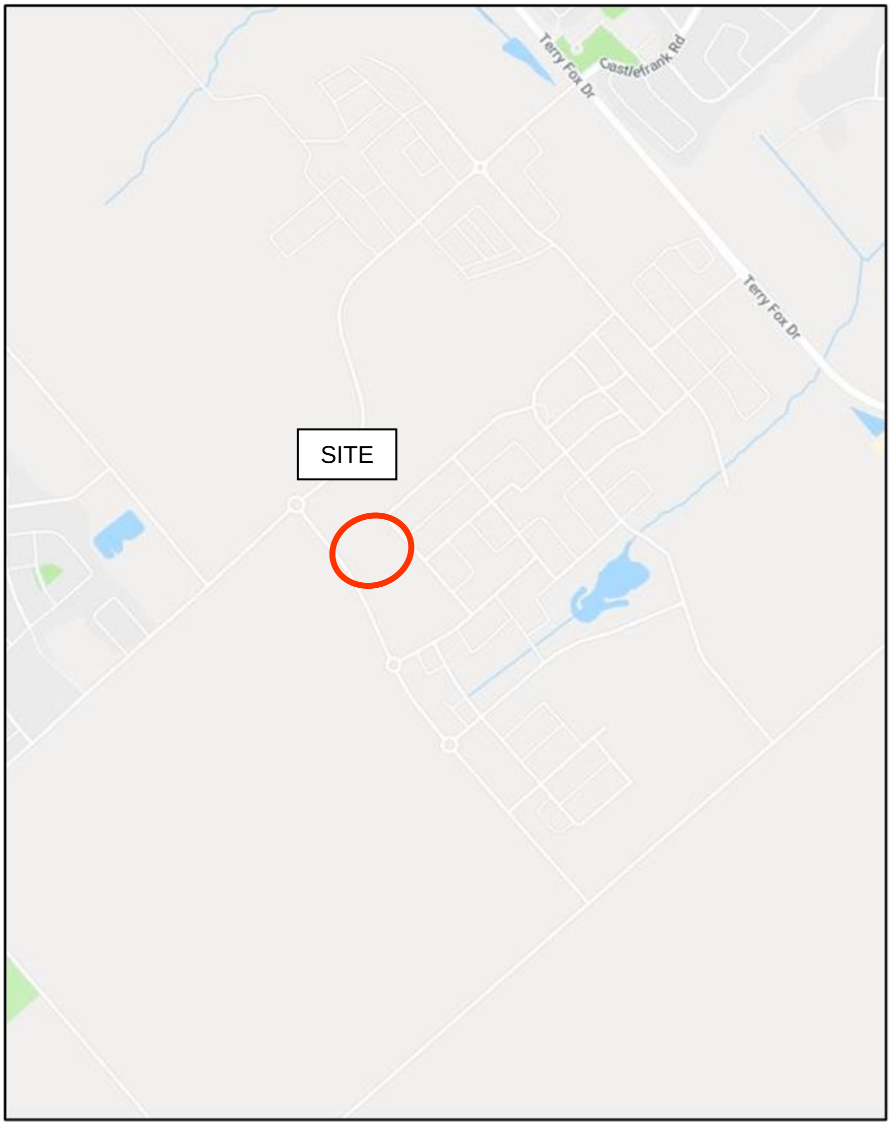
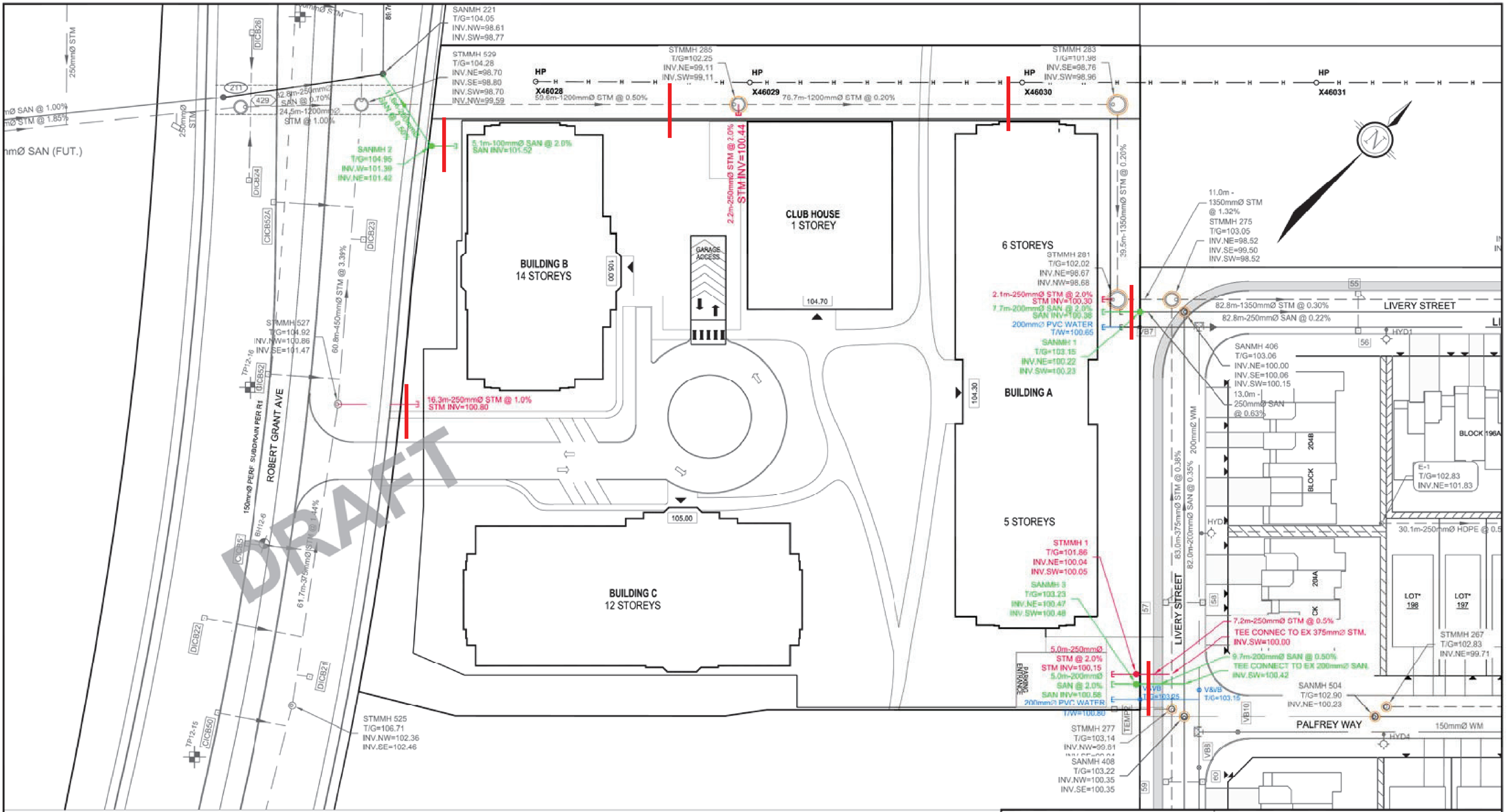


FIGURE 1

KEY PLAN



LEGEND

STMMH — PROPOSED STORM SEWER AND MANHOLE

SANMH — PROPOSED SANITARY SEWER AND MANHOLE

V&VB — PROPOSED WATERMAIN AND VALVE & VALVEBOX

STMMH — *PROPOSED STORM SEWER AND MANHOLE*

SANMH — *PROPOSED SANITARY SEWER AND MANHOLE*

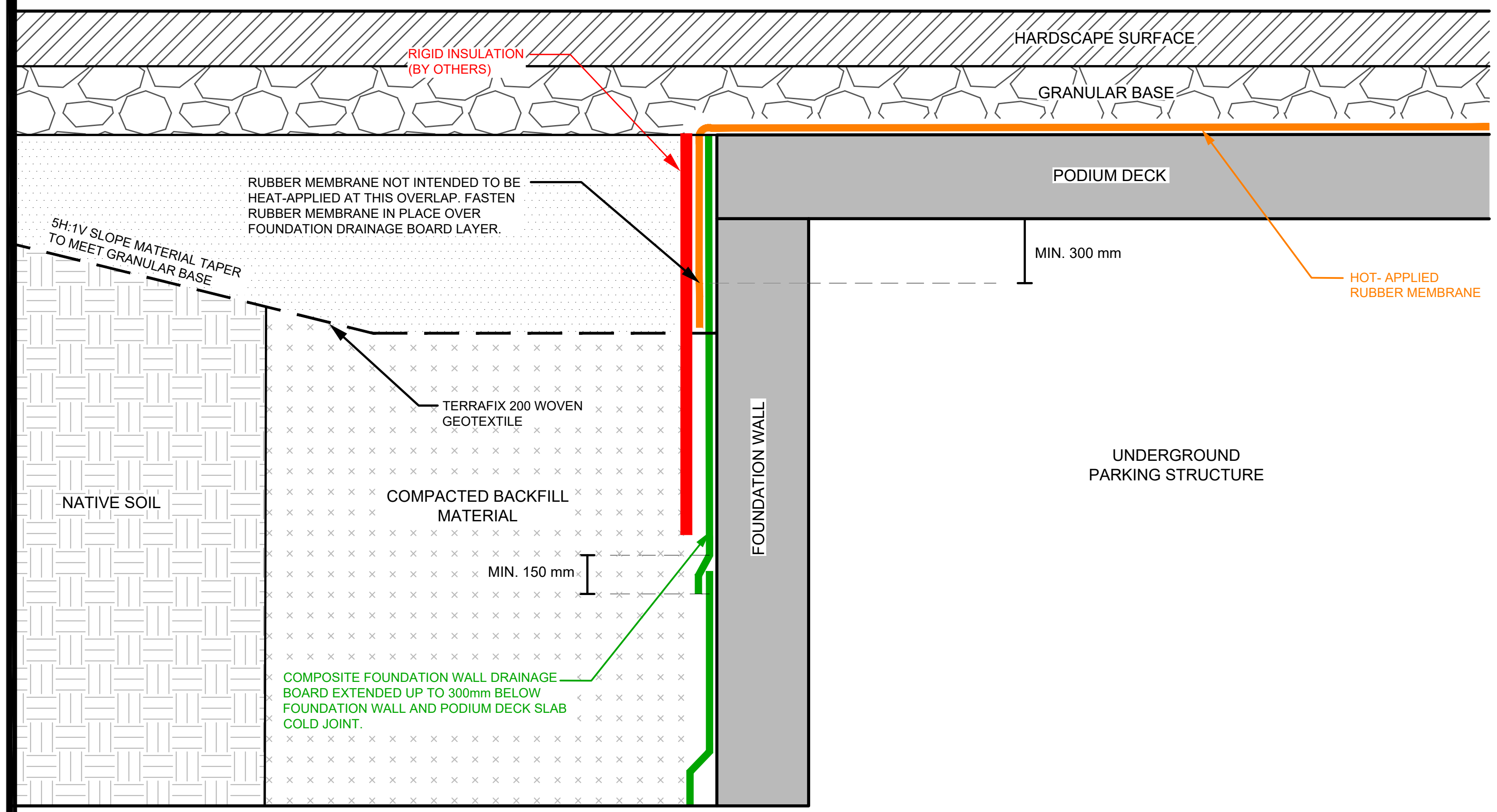
V&VB — *PROPOSED WATERMAIN AND VALVE & VALVEBOX*

/ Proposed Clay Seal Locations

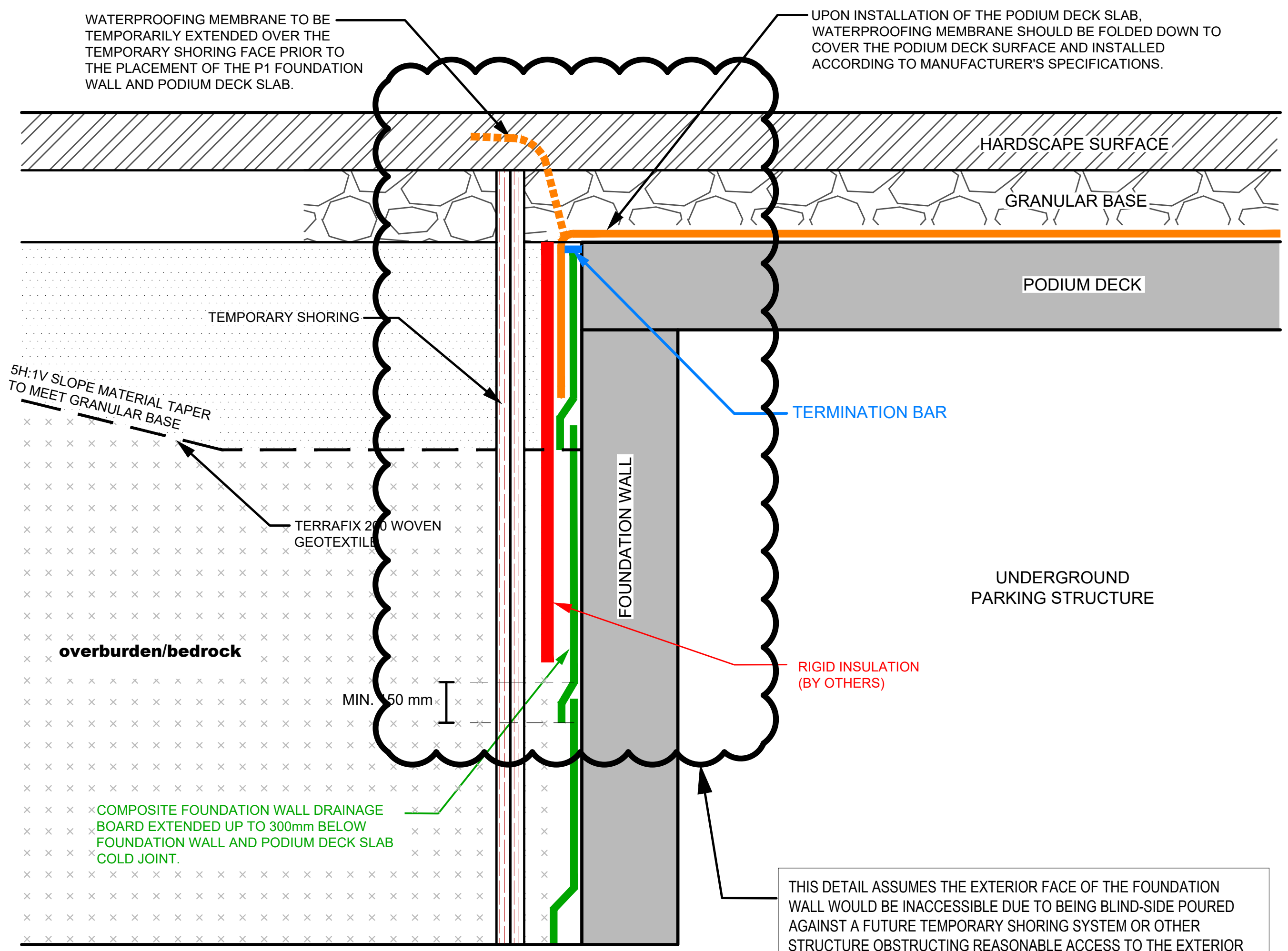
Figure 2 - Proposed Clay Seal Locations

SHT11X17.DWG - 279mmX432mm

OPTION A - DOUBLE-SIDE POURED
TOP OF FOUNDATION WALL



OPTION B - BLIND-SIDE POURED
TOP OF FOUNDATION WALL



THIS DETAIL ASSUMES THE EXTERIOR FACE OF THE FOUNDATION WALL WOULD BE INACCESSIBLE DUE TO BEING BLIND-SIDE POURED AGAINST A FUTURE TEMPORARY SHORING SYSTEM OR OTHER STRUCTURE OBSTRUCTING REASONABLE ACCESS TO THE EXTERIOR FACE OF THE CONCRETE WALL. IF THIS CONDITION IS OBSERVED THROUGHOUT THE SUBJECT SITE, THE TIE-IN DETAIL DEPICTED HEREIN SHOULD BE IMPLEMENTED AND REVIEWED IN THE FIELD BY PATERSON PERSONNEL AT THE TIME OF CONSTRUCTION.

NOTES:

THE ABOVE DETAIL FOR HOT RUBBER AND DRAINAGE BOARD OVERLAP IS APPLICABLE TO ALL EDGE-PORTIONS OF THE PODIUM DECK AND/OR SUSPENDED GROUND FLOOR SLAB STRUCTURE.

APPLICABILITY THICKNESS AND EXTENSIONS OF RIGID INSULATION ARE SPECIFIED BY OTHERS

WHERE THE GRADING SURFACE TERMINATES AGAINST THE BUILDING FACE AND PAVEMENT STRUCTURE IS NOT LOCATED ABOVE THE EDGE OF THE FOUNDATION WALL AND PODIUM DECK SLAB AS DEPICTED HEREIN, IT IS RECOMMENDED TO PROVIDE A SUITABLE TERMINATION BAR TO SEAL THE TOP ENDLAP OF THE HOT-APPLIED RUBBER MEMBRANE LAYER TO THE VERTICAL FACE OF THE STRUCTURE. THIS WOULD BE REQUIRED TO MITIGATE THE POTENTIAL FOR THE MIGRATION OF WATER BEHIND THE RUBBER MEMBRANE.

ALL PORTIONS OF THE ABOVE-NOTED DETAIL (INSULATION OF FOUNDATION DRAINAGE BOARD, TERMINATION BAR, HOT-RUBBER MEMBRANE OVER SLAB, FOUNDATION WALL CONSTRUCTION JOINT AND OVERLAPPING/SHINGLING OF DRAINAGE BOARD) SHOULD BE REVIEWED AT THE TIME OF CONSTRUCTION BY PATERSON PERSONNEL.

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
0			

CANADIAN RENTAL DEVELOPMENT SERVICES
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI STOREY BUILDINGS
OTTAWA, 5000 ROBERT GRANT AVENUE ONTARIO
Title:
PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

Scale: N.T.S
Date: 09/2023
Drawn by: RCG
Report No.: PG4562-1
Checked by: ZA
Dwg. No.: FIGURE 3
Approved by: MS
Revision No.: 0

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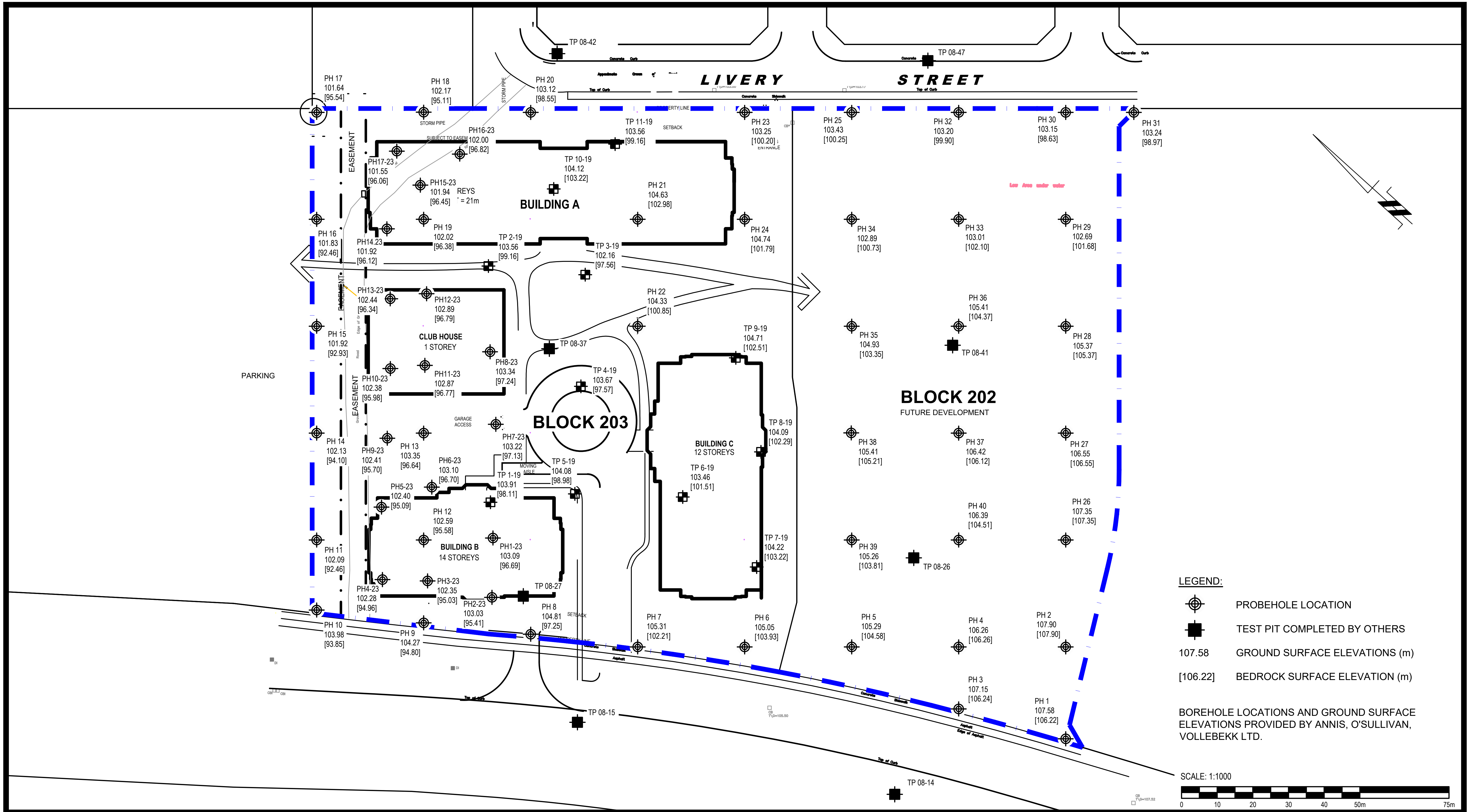

LEGEND:

⌖ PROBEHOLE LOCATION

■ TEST PIT COMPLETED BY OTHERS

107.58 GROUND SURFACE ELEVATIONS (m)

[106.22] BEDROCK SURFACE ELEVATION (m)

BOREHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
5	2023 PROBEHOLES ADDED TO PLAN	30/08/2023	MS
4	BASE PLAN UPDATED	26/03/2019	FA
3	2019 TEST PITS ADDED TO PLAN	25/02/2019	NC
2	TEST PITS BY OTHERS ADDED TO PLAN AND UPDATED BASE PLAN	09/01/2019	FA
1	UPDATED TO LATEST BASE PLAN	07/12/2018	FA

CANADIAN RENTAL DEVELOPMENT SERVICES
GEOTECHNICAL INVESTIGATION
PROP. DEVELOPMENT - FERNBANK CROSSING - CROSSING AVE.
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:1000
Date: 11/2018
Drawn by: RCG
Report No.: PG4562
Checked by: FA
Dwg. No.: PG4562-1
Approved by: DJG
Revision No.: 5

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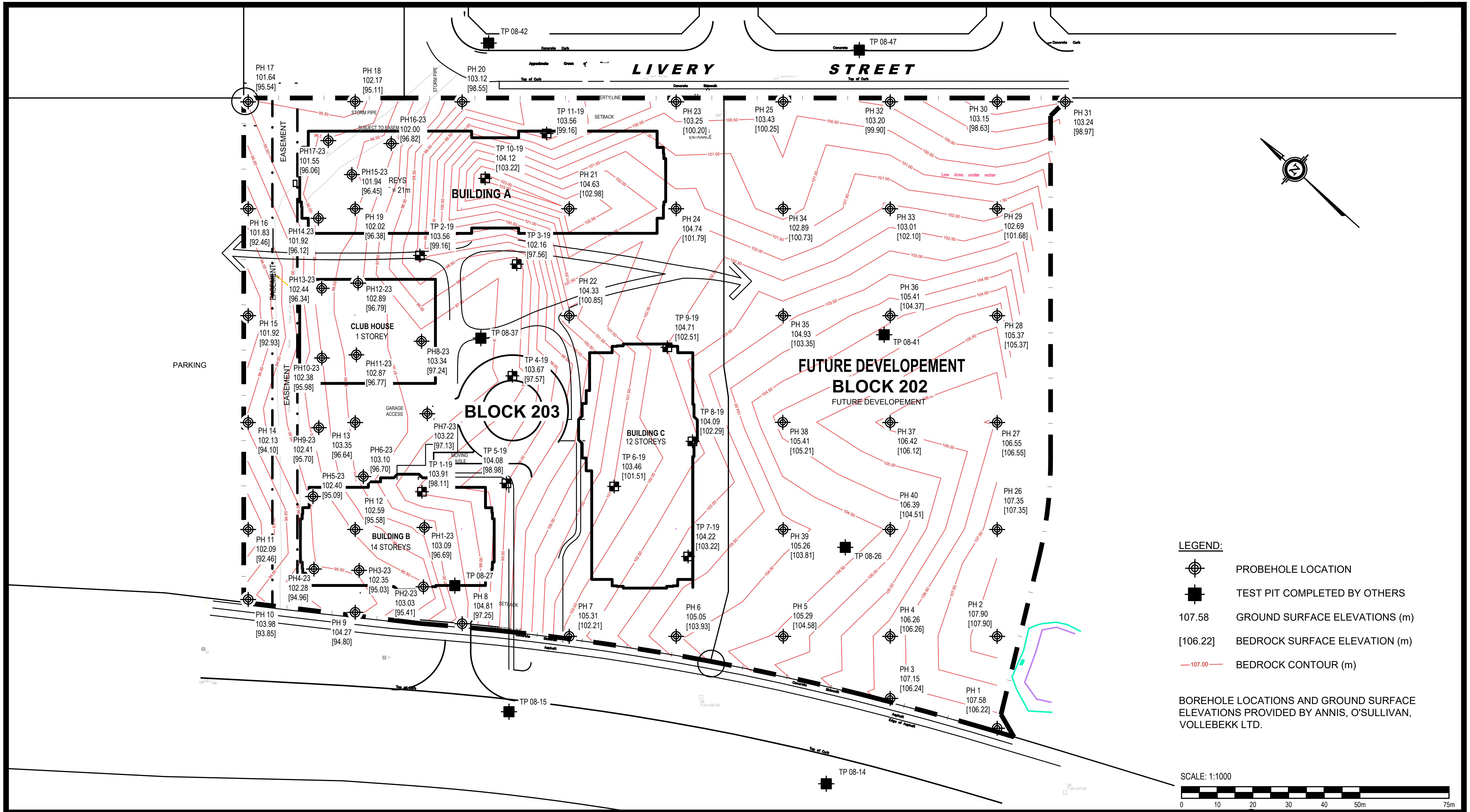

LEGEND:

⌖	PROBEHOLE LOCATION
■	TEST PIT COMPLETED BY OTHERS
107.58	GROUND SURFACE ELEVATIONS (m)
[106.22]	BEDROCK SURFACE ELEVATION (m)
—107.00—	BEDROCK CONTOUR (m)

BOREHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
5	SUPPLEMENTAL PROBEHOLES	08/30/2023	MS
4	BASE PLAN UPDATED	26/03/2019	FA
3	2019 TEST PITS ADDED TO PLAN	25/02/2019	NC
2	TEST PITS BY OTHERS ADDED TO PLAN AND UPDATED BASE PLAN	09/01/2019	FA
1	UPDATED TO LATEST BASE PLAN	07/12/2018	FA

CANADIAN RENTAL DEVELOPMENT SERVICES

GEOTECHNICAL INVESTIGATION

PROPOSED MULTISTOREY BUILDINGS

OTTAWA, 5000 ROBERT GRANT AVENUE ONTARIO

Title:

BEDROCK CONTOUR PLAN

Scale: 1:1000	Date: 11/2018
Drawn by: RCG	Report No.: PG4562
Checked by: ZA	Dwg. No.: PG4562-2
Approved by: MS	Revision No.: 5

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