PATERSON GROUP

Geotechnical Investigation

Proposed Hi-Rise Buildings

Towers 5 and 6 – Petrie's Landing
8900 Jeanne D'Arc Boulevard
Ottawa, Ontario

Prepared for 6382983 Canada Inc.

Report PG3908 – 3 Revision.04 dated November 28, 2025

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by 6382983 Canada Inc. (Brigil) to carry out a geotechnical investigation for the proposed high-rise buildings (Tower 5 and 6) to be located within the complex at 8900 Jeanne D'Arc Boulevard, in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the current this geotechnical investigation was to:

❑ Determine the subsoil and groundwater conditions at this site by means of test holes.

❑ 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. 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.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation.

2.0 Proposed Development

Based on available information, the proposed Towers 5 and 6 will consist of high-rise residential buildings. It is understood that 4 levels of underground parking are proposed for Towers 5 and 6. The parking garage will extend beyond the footprint of each tower structure. The development will also include associated asphaltic parking areas, access lanes and landscaped areas.

It is further anticipated that the site will be fully municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on July 16 to 19, 2018 and consisted of a total of 9 boreholes sampled to a maximum depth of 15.9 m below the existing grade throughout the subject site. Three boreholes were located in proximity to the proposed towers 5 and 6. A dynamic cone penetration test (DCPT) was carried out at each borehole to determine inferred bedrock depth which ranged from 22.8 to 32 m below the existing grade. The borehole locations for the current investigation were determined in the field by Paterson personnel taking into consideration existing borehole coverage and existing site features. The locations of the boreholes are illustrated on Drawing PG3908-3 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a track-mounted auger 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 testing procedure for boreholes consisted of augering to the required depths and at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Borehole samples were recovered from a 50 mm diameter split-spoon (SS) or the auger flights (AU). All soil samples were visually inspected and initially classified on site. The split-spoon and auger samples were placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the split-spoon and auger samples were recovered from the test holes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

The thickness of the overburden was evaluated during the course of the investigation by a dynamic cone penetration test (DCPT) at all borehole locations. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Undrained shear strength testing was carried out at regular depth intervals in cohesive soils. This testing was done in general accordance with ASTM D2573-08 Standard Test Method for Field Vane Shear Test in Cohesive Soil. Reference should be made to the Soil Profile and Test Data Sheets provided in Appendix 1.

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

Groundwater

A piezometer was installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The groundwater level readings are presented on the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations were determined by Paterson field personnel. The 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 are referenced to a geodetic datum. The location of the boreholes and the ground surface elevation at each borehole location are presented on Drawing PG3908-3 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were collected from the subject site during the investigation and were visually examined in our laboratory to review the results of the field logging. All samples will be stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless otherwise directed.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The majority of the subject site consists of vacant land and is relatively flat. Small piles of fill and a construction access road to Tower 3 and 4 are located on the subject site. The ground surface within the subject site slopes gradually towards the north and northeast portion of the site. It should be noted that the subject site is bordered from the North and Northeast by an existing residential high-rise building (Tower 1 and 2) and the associated above ground parking areas.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of topsoil underlain by a fill consisting of silty sand mixed with clay and/or gravel. A very stiff brown silty clay deposit extending to depths over 15 m below the existing grade. Practical refusal to DCPT was encountered in all boreholes between 22.8 and 27.1 m depth below existing grade. Specific details of the soil profile at each test hole location are presented Appendix 1.

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and dolomite of the Gull River formation. The overburden drift thickness is estimated to be between 20 to 35 m.

4.3 Groundwater

Based on Paterson's review of the groundwater measurements collected on July 24, 2018, the groundwater is expected to be at a depth ranging between 5 to 6 m below existing ground surface. It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled boreholes. Groundwater conditions can also be estimated based on the observed colour, moisture levels and consistency of the recovered soil samples.

5.0 Discussion

5.1 Geotechnical Assessment

Foundation Design Considerations

From a geotechnical perspective, the subject site is suitable for the proposed high-rise buildings (Towers 5 and 6). It is expected that the proposed high-rise buildings will be founded on end bearing piled foundations extending to the inferred limestone bedrock surface. It is also expected that the underground parking levels will be founded on conventional spread footings or raft slab placed on an undisturbed very stiff silty clay deposit bearing surface.

A control joint between the piled foundation and the underground parking foundation can be considered to avoid differential settlement. The structural design will dictate if this is required.

Permissible Grade Raise

Due to the presence of a silty clay layer, the subject site is subjected to a permissible grade restriction. Our permissible grade raise recommendations are discussed in Subsection 5.3. Based on this review and the proposed subsurface profile, a permissible grade raise restriction of 3 m will be assigned for the subject site.

The above and other considerations are discussed in the following paragraphs.

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 and other settlement sensitive structures. The existing fill material, where free of organic materials, should be reviewed by the geotechnical consultant at the time of construction to determine if the existing fill can be left in place below paved areas and below the slab granular fill layers.

Fill Placement

Fill placed for grading beneath the building area should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill 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 the material's standard Proctor maximum dry density (SPMDD).

Site-excavated soil can be placed as general landscaping fill where settlement is a minor concern of the ground surface. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm thick lifts and to a minimum density of 95% of the respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls due to the frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

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 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 95% of its SPMDD.

Compact Granular Fill Working Platform for Pile Installations

Should proposed structures be supported on a pile foundation, the use of heavy equipment would be required to install the piles (i.e., pile driving crane). It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site freely without sinking in the clay and causing significant disturbance to the underlying soil.

A typical working platform could consist of 600 mm of OPSS Granular B, Type II crushed stone placed and compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in maximum 300 mm thick lifts. However, Paterson should review the size and expected load of equipment to be used prior to arrival at the site.

Protection of Subgrade

Once the pile driving is completed the base of the excavation should be covered with a minimum 100 - 125 mm thick lean concrete mud slab. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic or workers and equipment. Additionally, the mudslab also acts as a hydraulic barrier and reduce the water infiltration through the subgrade.

For excavations in clay, the final excavation of the bearing surface level and the placement of the mud slab should be completed in smaller sections to avoid exposing large areas of the silty clay to potential disturbances due to drying as a result of evaporation of the moisture within the upper silty clay layer.

5.3 Foundation Design

Conventional Shallow Footings

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over an undisturbed, very stiff silty clay bearing surface from an elevation 51.0 m to 45.0 m can be designed using 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, placed over an undisturbed, very stiff silty clay bearing surface from an elevation 45.0 m to 38.0 m can be designed using bearing resistance value at serviceability limit states (SLS) of **175 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **250 kPa**.

A geotechnical resistance factor of 0.5 was applied to the reported 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 prior to the placement of concrete for footings.

For the parking garage, the bearing resistance value given for footings at SLS will be subjected to potential post construction total and differential settlements of 20 and 10 mm, respectively.

Raft Foundation

The following parameters may be used for raft design and will apply for an undisturbed soil bearing surface. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings and approved by the geotechnical consultant.

For design purposes, it was assumed that the base of a raft foundation would be located between the geodetic elevation of 41.0 to 38.0 m on a very stiff silty clay bearing surface. The contact pressure provided considers the stress relief associated with the soil removal required for 4 levels of underground parking.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. 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 bearing resistance value at SLS (contact pressure) of **300 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 the proposed building. The factored bearing resistance value at ULS (contact pressure) can be taken as **400 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS. For this case, the modulus of subgrade reaction was calculated to be **20.0 MPa/m** for a contact pressure of **300 kPa**.

Based on the following assumptions for the raft foundation, the proposed building can be designed using the above parameters with a total and differential settlement of 25 and 15 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. Above the groundwater level, adequate lateral support is provided to a stiff silty clay when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:1V passes only through in situ soil or engineered fill.

Below the groundwater level, adequate lateral support is provided to stiff silty clay bearing media when a plane extending down and out from the bottom edges of the footing, at a minimum of 1.5H:1V, passes only through the in-situ soil or engineered fill of the same or higher capacity as that of the bearing medium.

End Bearing Piled Foundation

It is expected that the main building will be constructed over concrete filled steel pipe piles driven to refusal on the bedrock surface.

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

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring.

For this project, the dynamic monitoring of two to four piles would be recommended for each tower. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 1 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	925	1110	6	27
245	11	1050	1260	6	31
245	13	1200	1440	6	35

Permissible Grade Raise Recommendations

Although no significant grade raises are expected for the subject development, the grade raise restriction for the subject site was calculated to be **3 m** above original ground surface. Generally, the potential long-term settlement is evaluated based on the compressibility characteristics of the silty clay. These characteristics have been conservatively estimated based on the shear strength of the clay and the subsoil condition observed at the test pit locations. It should be noted that a post-development groundwater lowering of 0.5 m was applied to the permissible grade raise restriction.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for the foundations considered. Due to the compactness of the silty clay deposit and the long-term groundwater level, soils underlying the subject site are not susceptible to liquefaction. Refer 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 areas 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.

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 Wall

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 $P_o = 0.5\ K_o\ \gamma\ H^2$, 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

Car only parking, access lanes, heavy truck parking areas, local and collector roadways are anticipated at this site. The proposed pavement structures are shown in Tables 2 and 3.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 3 - Recommended 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, in situ soil, select subgrade material or OPSS Granular B Type I or II material placed over in situ soil or fill	

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 Type II material.

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

Pavement Structure Drainage

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

Where silty clay is encountered at subgrade level, consideration should be given to installing subdrains during the pavement construction. These drains should be constructed according to City of Ottawa specifications. The drains should be connected to a positive outlet. The subgrade surface should be crowned to promote water flow to the drainage lines. The subdrains will help drain the pavement structure, especially in early Spring when the subgrade is saturated and weaker and, therefore, more susceptible to permanent deformation.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

Based on the preliminary information provided, it is expected that a portion of the proposed building foundation walls located below the long-term groundwater table. To limit long-term groundwater lowering, it is recommended that a groundwater infiltration control system be designed for the proposed building.

The groundwater infiltration control system should consist of the following:

- A suitable waterproofing membrane such as a Colphene BSW or equivalent, installed over the composite drainage board against the temporary shoring system. The waterproofing membrane should start approximately 300 mm above the proposed P1 Level and extend to the bottom of the footing. The waterproofing membrane should also extend a minimum of 300 mm horizontally below the underside of footing.
- A composite drainage layer, such as Delta Drain 6000 or equivalent, installed against the temporary shoring system, behind the waterproofing membrane. The composite drainage layer should extend from finished grade to underside of footing level.
- A dampproofing membrane or a waterproofing membrane should be installed from the finished grade to the proposed P1 Level, overlapping a minimum of 300 mm onto the underlying waterproofing membrane.
- The foundation wall should be poured against the waterproofing membrane and/or dampproofing membrane.

Reference should be made to Figure 5 – Foundation Drainage System in Appendix 2.

Underfloor Drainage

Underfloor drainage will be required to control water infiltration below the lowest underground parking level slab that breaches the horizontal hydraulic barrier (minimum 100 to 125 mm thick concrete mud slab). The subfloor drainage system should be at the elevation of the foundation wall and no lower than 0.5 m from the top of the pile caps. The system can be placed near level (or with minimum slope 0.1%) with the sump pit(s).

Hydrostatic effect

With the proposed waterproofing layout, the structural design should consider partial hydrostatic loading on the foundation wall. It is recommended to assume 3 m water column (approximately 30 kPa) starting from the fourth underground slab level.

Elevator Pit Waterproofing

It is recommended that a horizontally applied Colphene BSW H waterproofing membrane (or approved other by Paterson) should be placed on an adequately prepared mud slab and extend vertically within the inside of the temporary forms of the elevator raft slab.

Once the concrete raft slab and elevator shaft sidewalls are poured in place, it is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved other) should be applied to the exterior of the elevator pit sidewalls. The Colphene Torch'n Stick waterproofing membrane should extend over the vertical portion of the previously applied Colphene BSW H waterproofing membrane installed on the concrete raft slab in accordance with the manufacturer's specifications. As a secondary defence, a continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the concrete raft slab below the elevator pit sidewalls.

A protection board should be placed over the waterproofing membrane to protect the waterproofing membrane from damage during backfilling operations. The area between the elevator pit and soil excavation face should be in-filled with lean concrete, OPSS Granular B Type 2 or Granular A crushed stone.

The above-noted efforts are recommended to be reviewed at the time of construction by Paterson personnel.

It should be noted that a waterproofed concrete (with Xypex Additive, or equivalent) is optional for this waterproofing option. Refer to Figure 4 - Elevator Pit Waterproofing, for specific details of the waterproofing.

Water Infiltration Volumes

During the construction phase, it's expected that water infiltration should have a steady state volume of less than 50,000 to 75,000 L/day plus any surface water infiltration following a precipitation event. The initial influx will be greater once the excavation extends below the long-term groundwater level. The zone of influence associated with the temporary dewatering during the construction excavation for 4 levels of underground parking will be approximately 12 m.

Provided that the recommendations detailed in subsection 6.1 are successfully implemented, groundwater infiltration rates of no greater than 25,000 L/day are expected. The zone of influence associated with the long-term dewatering at post construction for 4 levels of underground parking will be less than 6 m.

Foundation Backfill

Where required, backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000

Imported granular materials, such as clean sand or OPSS Granular B Type I or OPSS Granular A granular material, should otherwise be used for this purpose.

Adverse Effects from Dewatering on Adjacent Structures

The temporary dewatering program during construction will have a limited zone of influence of less than 12 m from the foundation perimeter and less than 6 m at post construction. The underlying native soil below the groundwater table at the subject site is a stiff silty clay deposit. The temporary dewatering of the silty clay deposit during the excavation and construction stage will not be susceptible to significant consolidation since the material is stiff to very stiff.

Implementation of the groundwater infiltration control system recommended above is expected to limit the drawdown of the local groundwater table over the long term and in a limited area. Therefore, in our opinion, no adverse effects to nearby structures and infrastructure are expected over the long term.

Flood Proofing

Based on the available information, the lower parking level of Tower 5A and 5B will be located below the 100-year flood level. However, the structure will be located more than 60 m away from the flood line and in a thick silty clay layer. Due to the low permeability of the silty clay layer and relatively short duration of flood events, no groundwater rise is expected from a major flood event near the proposed underground structure.

Paterson should complete regular site visit during construction to review the installation and placement of the drainage and waterproofing system.

6.2 Protection of Footings 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.

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 structure is 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 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. Generally, tie backs extend into bedrock and are grouted in place with minimum 30 or 40 MPa non shrink grout. 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 4 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m^3	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 is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component. For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

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

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce the potential 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 SPMDD.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, sub bedding and cover material. The barriers should consist of relatively dry and compatible brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be low through the sides of the excavation 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.

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 Persons as stipulated under O.Reg. 63/16.

Long-Term Groundwater Control

The recommendations for the proposed building long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building sub-slab drainage system will be directed to the proposed building cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, the groundwater flow should be low (i.e.- less than 25,000 L/day per building). A more accurate estimate can be provided at the time of construction once groundwater infiltration levels are observed. The groundwater flow should be controllable using conventional open sumps.

Impacts on Neighboring Structures

Based on observations, the long-term groundwater level is anticipated at depths below 10 m. A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. The extent of any significant groundwater lowering should occur within a limited range of the subject site due to the minimal temporary groundwater lowering.

The neighboring structures are expected to be founded within the brown silty clay crust bearing surface. No issues are expected, with respect to groundwater lowering, that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Retaining Wall Design

The retaining wall should be designed in accordance with the most recent Canadian Highway and Bridge Design Code (CSA S6:19). A live load equivalent to 17 kPa should be considered for traffic loading above the retaining wall. Consideration should be given to additional lateral loading exerted by wall features such as fencing and/or any other structures proposed to be placed along the high side of the retaining wall.

The proposed retaining wall was design as part of this review, a typical cross section for the wall is presented in Figure 2 in Appendix 2.

Lateral Earth Pressures

It is recommended that a minimum of 1 m of the backfill material to consist of clean imported engineered crushed stone such as OPSS Granular A or Granular B Type II. The soil parameters presented in Tables 5 and 2 should be used for the design of the retaining walls.

Table 5 - Geotechnical Parameters for backfill material

Material Description	Unit Weight (kN/m^3)		Friction Angle (°)	Friction Factor,	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'	φ'	tan δ	Active K_a	At-Rest K_o	Passive K_P
OPSS Granular A Fill (Crushed Stone)	22	13.5	36	0.6	0.26	0.41	3.85
OPSS Granular B Type II Fill (Crushed Stone)	22	14	38	0.6	0.24	0.38	4.2
Silty Clay	17	9	33	0.3	0.30	0.45	3.4

Notes:

I. Properties for fill materials are for condition of 98% of standard Proctor maximum dry density.
II. The earth pressure coefficients provided are for horizontal backfill profile.
III. For soil above the groundwater level the "drained" unit weight should be used and below groundwater level the "effective" unit weight should be used.

Drainage

A 100 mm diameter perforated drainage pipe wrapped in geotextile and surrounded on all sides by 150 mm of clear crushed stone, should be installed at the heel of the bottom block. The drainage should have positive drainage to a nearby outlet such as a catch basin or an existing ditch. It is recommended that the outlets be spaced evenly along the retaining wall with a minimum spacing of 30 m center to center passing through the wall or connected to a nearby catch basin.

Backfill Material

The retaining wall should be backfilled with free-draining granular backfill materials and incorporate longitudinal drains and weep holes to provide positive drainage of the backfill. For the purpose of this report, it is recommended that the wall be backfilled with either OPSS Granular B Type II or Granular A materials. The backfill should be placed within a wedge-shaped zone defined by a line drawn up and back from the back edge of the base block of the wall at an inclination of 1H:1V or a minimum of 1 m behind the back of the blocks. All material should be compacted to a minimum of 98% of the material's SPMDD.

Global Stability Analysis

The global stability analysis was modeled in Fine Geo 5, a computer program which permits a two-dimensional slope stability analysis calculating several methods including the Bishop's method, which is a widely accepted slope analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to forces favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsurface soil and groundwater conditions, a factor of safety greater than 1.0 is generally required for the failure risk to be considered acceptable.

A minimum factor of safety of 1.5 is generally recommended for conditions where the slope failure would comprise permanent structures. An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16 g was considered for the sections for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The retaining wall section was reviewed using the design loading according to CHBDC 2015.

The highest retaining wall cross-section was studied as the worst-case scenario and presented in Appendix 2.

Analysis Results

The factor of safety for the retaining wall section was greater than 1.5 for static conditions. Similarly, the results under seismic loading yielded a factor of safety for this section greater than 1.1.

Based on these results, the retaining wall is acceptable from a geotechnical perspective.

6.7 Winter Construction

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

The subsurface conditions mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and tarpaulins or other suitable means.

The base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions.

6.8 Corrosion Potential and Sulphate

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

7.0 Recommendations

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

- ❑ Review of the site master grading plan, once available.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Observation of the placement of the foundation insulation, if applicable.
- ❑ Observe and review the installation of the drainage and waterproofing system.
- ❑ 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 the construction 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 and our recommendations when the drawings and specifications are complete.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than 6382983 Canada Inc. or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
November 28, 2025
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Pratheep Thirumoolan, M.Eng.

Joey R. Villeneuve, M.A.Sc., P.Eng, ing.

Report Distribution:

- ❏ 6382983 Canada Inc. (Brigil Construction)
- ❏ Paterson Group Inc

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - Petrie's Landing I
100 Inlet Private, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE July 16, 2018

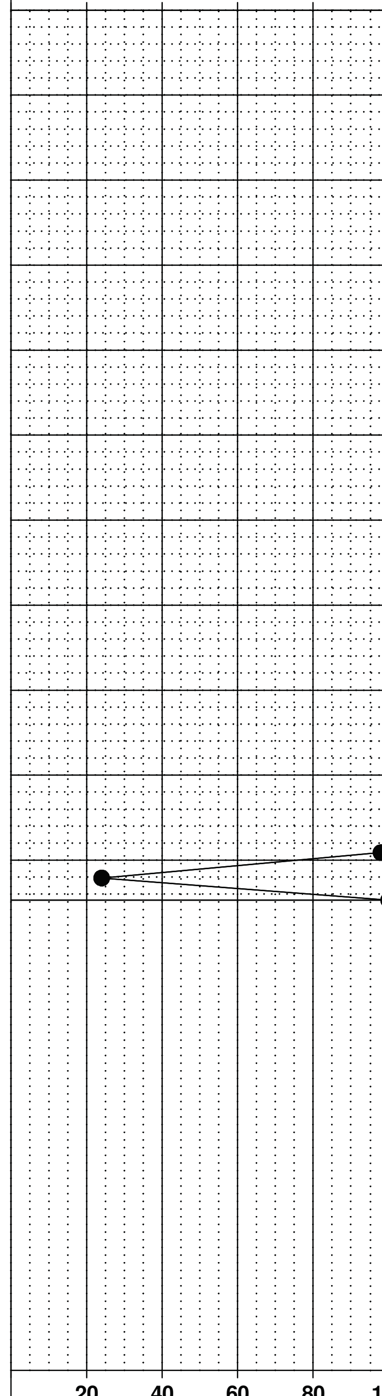
FILE NO. PG3908

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE: TYPE	SAMPLE: NUMBER	SAMPLE: % RECOVERY	SAMPLE: N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
Dynamic Cone Penetration Test commenced at 15.85m depth. Cone pushed to 25.3m depth.						16	37.46
						17	36.46
						18	35.46
						19	34.46
						20	33.46
						21	32.46
						22	31.46
						23	30.46
						24	29.46
						25	28.46
26.47						26	27.46
End of Borehole Practical DCPT refusal at 26.47m depth. (GWL @ 3.24m - July 24, 2018)							

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

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - Petrie's Landing I
100 Inlet Private, Ottawa, Ontario

DATUM Geodetic

REMARKS

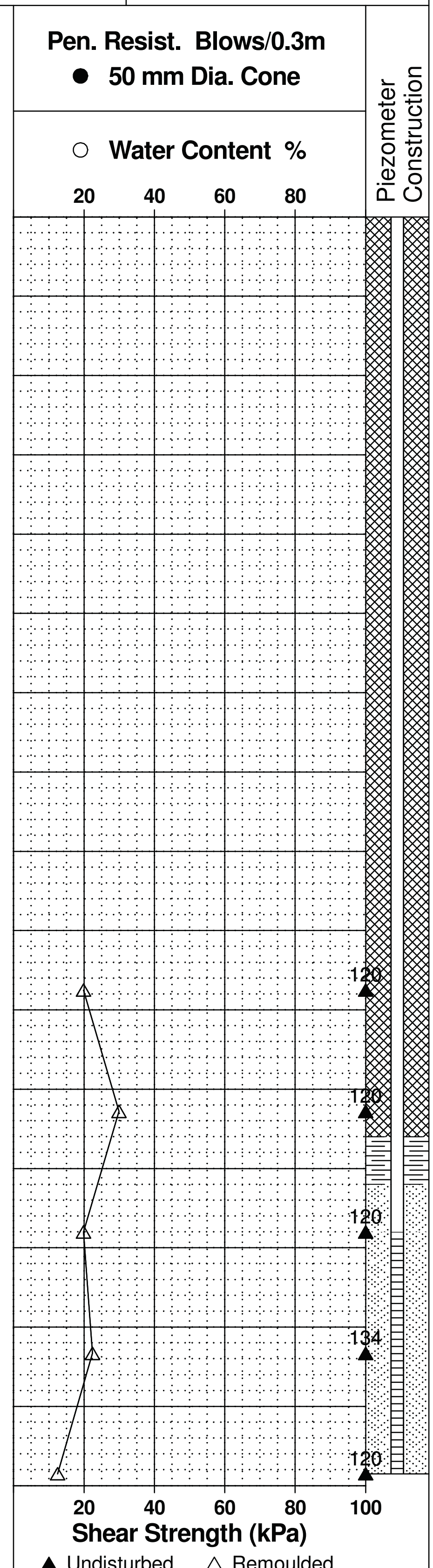
BORINGS BY CME 55 Power Auger

DATE July 13, 2018

FILE NO. PG3908

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
FILL: Brown silty sand with crushed stone and organics (0.48)		AU	1			0	54.12
						1	53.12
						2	52.12
						3	51.12
						4	50.12
						5	49.12
Very stiff, brown **SILTY CLAY**						6	48.12
- grey by 6.3m depth						7	47.12
						8	46.12
						9	45.12
						10	44.12
						11	43.12
						12	42.12
						13	41.12
						14	40.12
						15	39.12
(15.85)						16	38.12



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - Petrie's Landing I
100 Inlet Private, Ottawa, Ontario

DATUM Geodetic

REMARKS

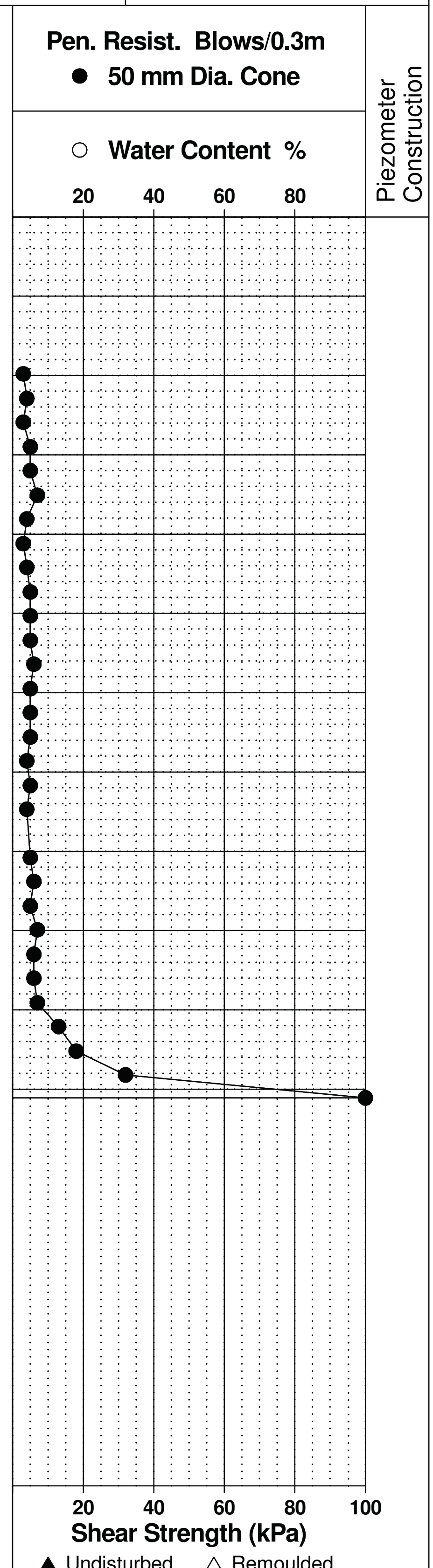
BORINGS BY CME 55 Power Auger

DATE July 13, 2018

FILE NO. PG3908

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
Dynamic Cone Penetration Test commenced at 15.85m depth. Cone pushed to 17.7m depth.						16	38.12
						17	37.12
						18	36.12
						19	35.12
						20	34.12
						21	33.12
						22	32.12
						23	31.12
						24	30.12
						25	29.12
						26	28.12
27.11						27	27.12
End of Borehole Practical DCPT refusal at 27.11m depth. (BH dry - July 24, 2018)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - Petrie's Landing I
100 Inlet Private, Ottawa, Ontario

DATUM Geodetic

REMARKS

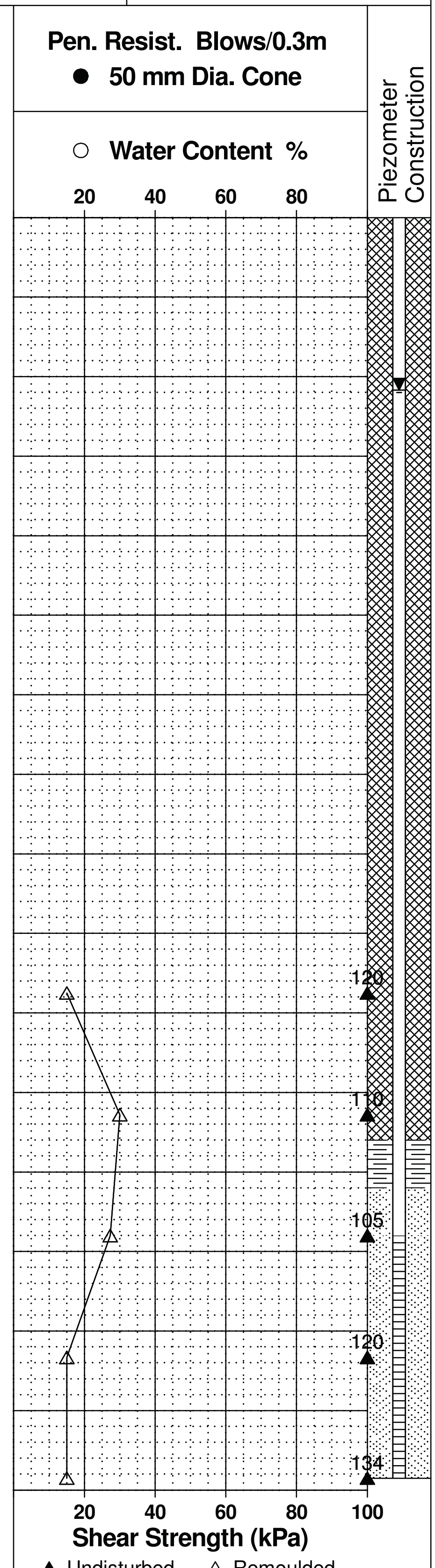
BORINGS BY CME 55 Power Auger

DATE July 16, 2018

FILE NO. PG3908

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	53.42
TOPSOIL 0.05		AU	1				
FILL: Gravel with sand 0.41							
						1	52.42
						2	51.42
						3	50.42
						4	49.42
						5	48.42
Very stiff, brown **SILTY CLAY**						6	47.42
- grey by 6.5m depth						7	46.42
						8	45.42
						9	44.42
						10	43.42
						11	42.42
						12	41.42
						13	40.42
						14	39.42
						15	38.42
15.85						16	37.42



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Development - Petrie's Landing I
100 Inlet Private, Ottawa, Ontario

DATUM Geodetic

REMARKS

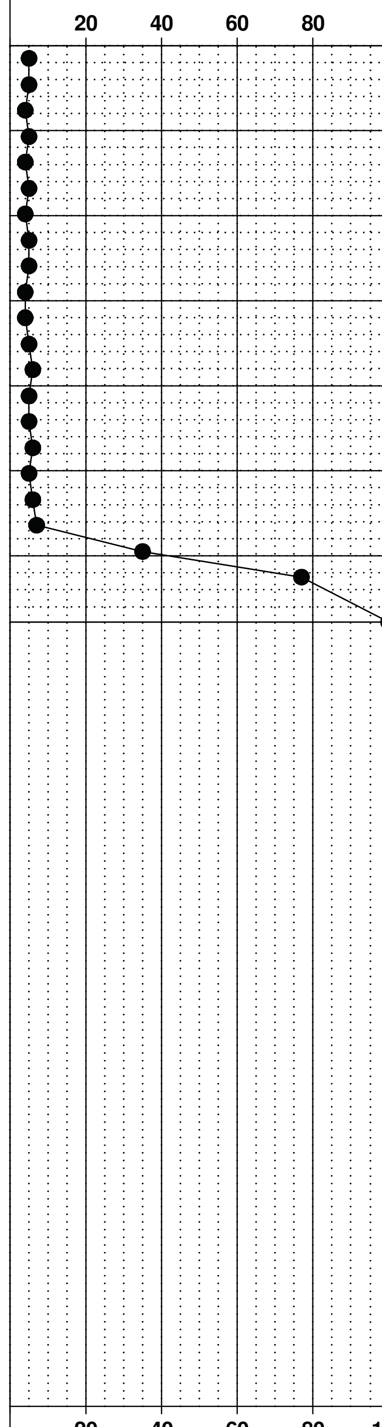
BORINGS BY CME 55 Power Auger

DATE July 16, 2018

FILE NO. PG3908

HOLE NO. BH 3

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 %	Piezometer Construction
GROUND SURFACE									
Dynamic Cone Penetration Test commenced at 15.85m depth.						16	37.42		
						17	36.42		
						18	35.42		
						19	34.42		
						20	33.42		
						21	32.42		
22.78						22	31.42		
End of Borehole Practical DCPT refusal at 22.78m depth. (GWL @ 2.17m - July 24, 2018)									



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 x 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

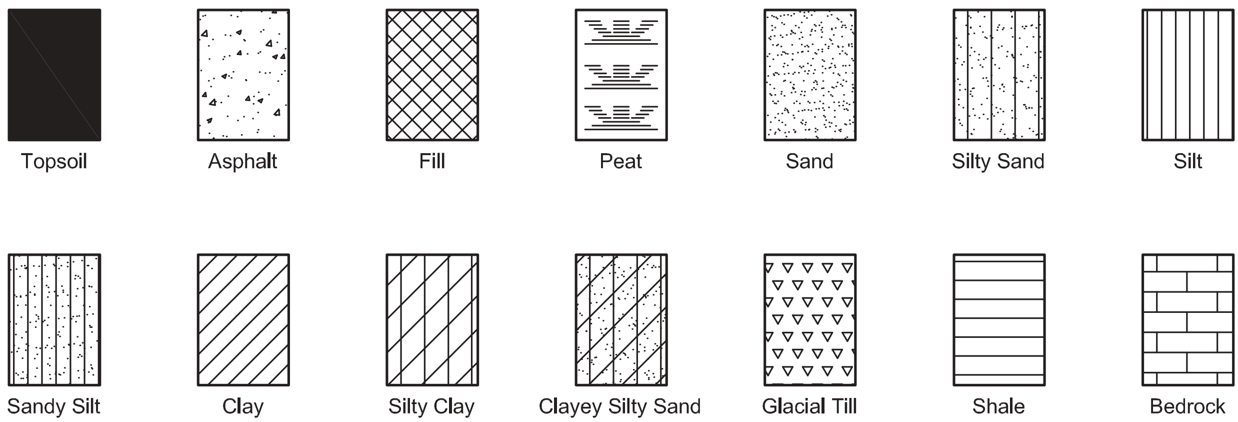
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

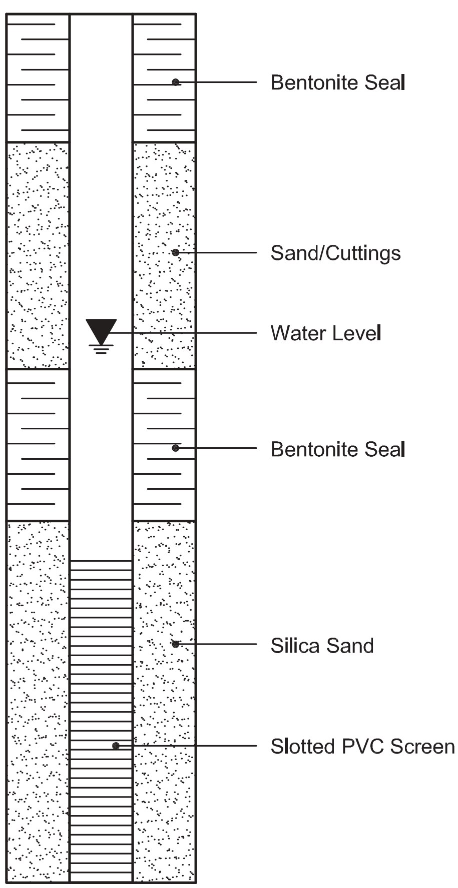
SYMBOLS AND TERMS (continued)

STRATA PLOT

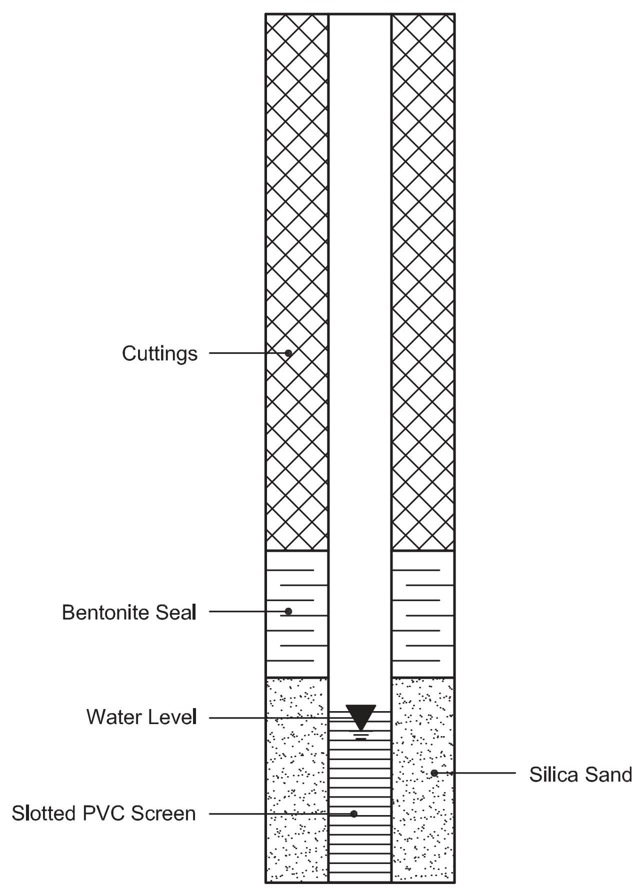


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



	Client ID:	BH-7	-	-	-
	Sample Date:	07/18/2018 09:00	-	-	-
	Sample ID:	1830026-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	76.1	-	-	-
General Inorganics					
pH	0.05 pH Units	6.82	-	-	-
Resistivity	0.10 Ohm.m	177	-	-	-
Anions					
Chloride	5 ug/g dry	10	-	-	-
Sulphate	5 ug/g dry	13	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - TYPICAL RETAINING WALL DETAIL

FIGURE 3 - FOUNDATION DRAINAGE SYSTEM

FIGURE 4 - ELEVATOR PIT WATERPROOFING

FIGURE 5 - SUBFLOOR DRAINAGE LAYOUT

DRAWING PG3098-3 - TEST HOLE LOCATION PLAN

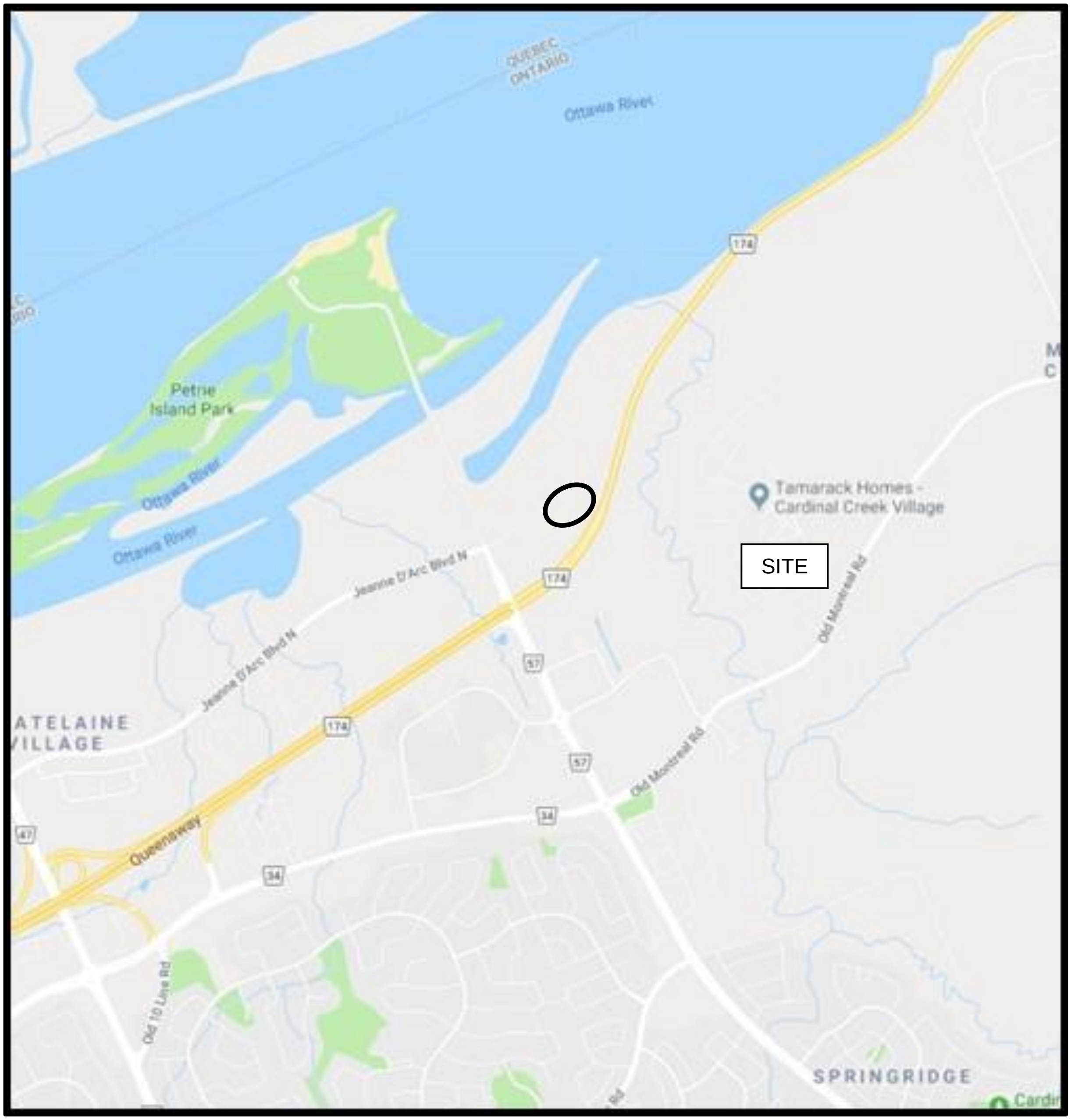


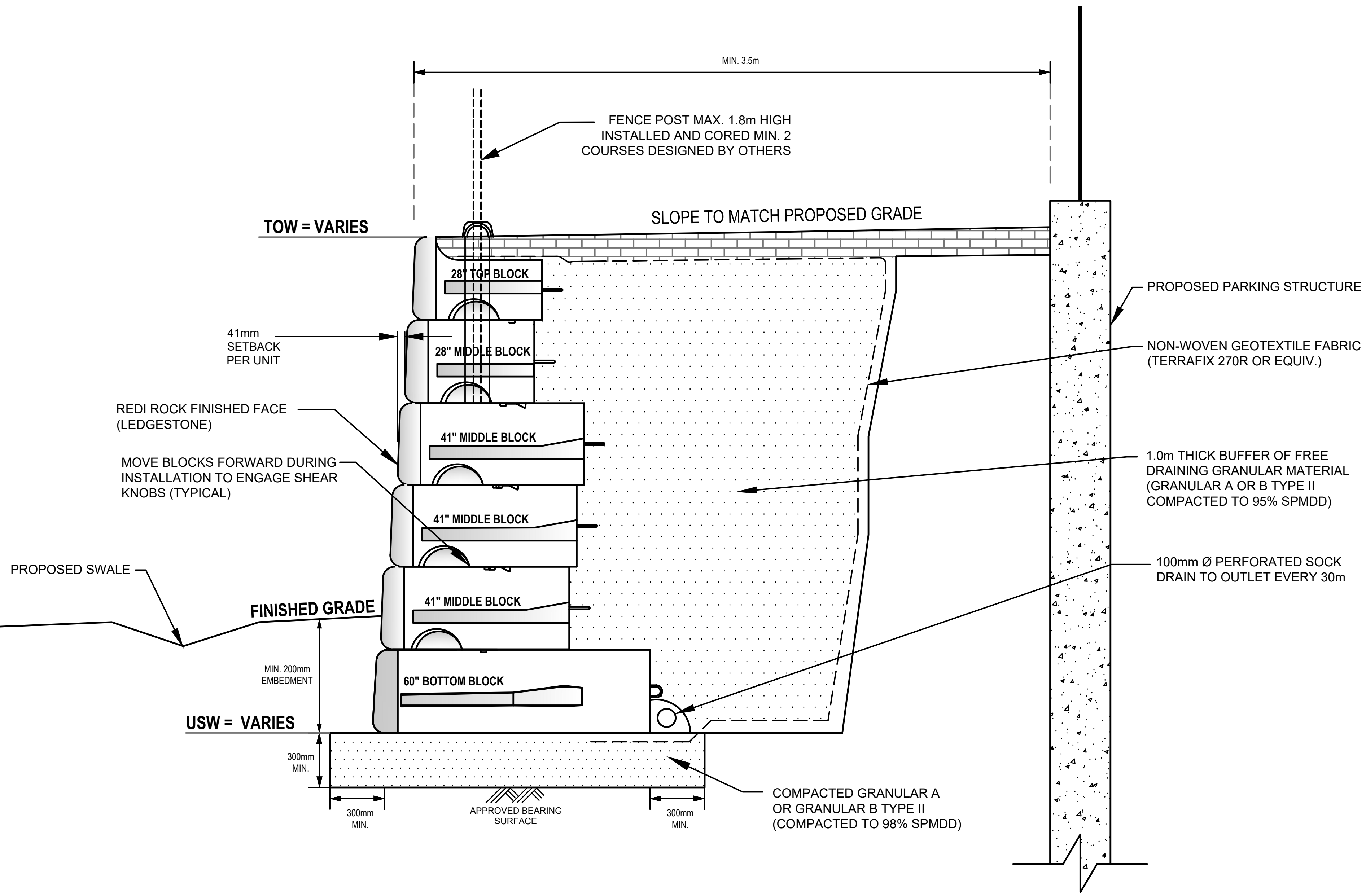
FIGURE 1

KEY PLAN

PATERSON GROUP

TYPICAL CROSS-SECTION:

SCALE 1:30



NOTES:

1. THE CONTRACTOR IS SOLELY RESPONSIBLE FOR UTILITY CLEARANCES AND CONSTRUCTION SITE SAFETY. PATERSON GROUP SHALL NOT BE RESPONSIBLE FOR MEANS OR METHODS OF CONSTRUCTION OR FOR SAFETY OF WORKERS OR OF THE PUBLIC.

2. THIS DESIGN IS BASED ON THE FOLLOWING SOIL PROPERTIES:

PROPERTY	RETAINED FILL	FOUNDATION MEDIUM (1)
FRICTION ANGLE - ϕ	36°	30°
UNIT WEIGHT - γ	22 kN/m3	17 kN/m3
COHESION - C	0	5 kPa
SOIL TYPE TYPE II	OPSS GRANULAR B	STIFF SILTY CLAY

MATERIAL PROPERTIES ARE BASED ON SITE EVALUATION BY PATERSON GROUP AND DISCUSSIONS WITH CONTRACTOR. SEISMIC LOADING WAS EVALUATED ACCORDING TO THE CURRENT CHBDC WITH A PEAK GROUND ACCELERATION VALUE OF 0.308.

3. THE DESIGN ELEVATIONS USED WERE BASED ON GRADING PLAN DRAWINGS PROVIDED BY BRIGIL. THE WALL BASE DESIGN ASSUMES A BEARING RESISTANCE AT SLS OF 150 kPa ON SILTY CLAY. PATERSON GROUP ENGINEER SHOULD OBSERVE THE BEARING CONDITIONS AND ADJUST THE THICKNESS OF THE GRANULAR BASE TO ACCOMMODATE THE SITE CONDITIONS, IF NECESSARY.

4. THE DESIGN HAS BEEN REVIEWED FOR THE STABILITY OF THE PRECAST MODULAR RETAINING WALL SYSTEM AND GLOBAL STABILITY WITH A FACTOR OF SAFETY OF 1.5 FOR STATIC CONDITIONS AND 1.1 UNDER SEISMIC CONDITIONS. WALL GEOMETRY AND GRADE ELEVATIONS ABOVE AND BELOW THE WALL SHOULD CONFORM WITH THE GRADING PLAN PROVIDED HERE IN. IF ACTUAL SITE GRADES VARY SIGNIFICANTLY FROM THOSE SHOWN OR IF THE BACK SLOPE DOES NOT CONFORM, INSTALLATION SHALL NOT PROCEED UNTIL THE DESIGN IS VERIFIED OR MODIFIED IN THE APPLICABLE AREA.

5. PRECAST UNITS SHALL BE REDI-ROCK RETAINING WALL UNITS MANUFACTURED UNDER LICENSE FROM REDI-ROCK.

6. THE WALL BASE FOR THE WALL SHALL CONSIST OF A MIN. 500mm THICK LAYER OF OPSS GRANULAR B TYPE II COMPACTED TO MIN. 98% OF THE MATERIAL'S SPMDD. THE ENGINEERED PAG LAYER SHOULD BE REINFORCED BY WRAPPING THE ENGINEERED FILL WITH BIAXIAL GEOGRID LINER SUCH AS TERRAFIX TBX2500 OR EQUIVALENT. THE GEOGRID SHOULD BE FOLLOWED BY NON-WOVEN GEOTEXTILE LINER SUCH AS TERRAFIX 270R OR EQUIVALENT TO PREVENT THE STONE FROM EXITING THROUGH THE GRIDS OF THE GEOGRID. THE GEOGRID AND GEOTEXTILE SHOULD BE WRAPPED A MINIMUM 1m BELOW THE BASE BLOCK AS SHOWN ON THE CROSS SECTION DETAIL. FOR WALL RR6, A REGULAR 300mm THICK LAYER OF OPSS GRANULAR A OR GRANULAR B TYPE II COMPACTED TO 98% OF THE MATERIAL'S SMPDD BELOW THE BASE BLOCK IS RECOMMENDED. THE SURFACE OF GRANULAR BASE MAY BE DRESSED WITH FINER AGGREGATE TO AID LEVELING. ENSURE GRADATION OF DRESSING MATERIAL IS SUCH AS TO PRECLUDE LOSS OF FINES INTO BASE. THE THICKNESS OF DRESSING LAYER SHOULD NOT EXCEED 3 TIMES THE MAXIMUM PARTICLE SIZE USED.

7. WALL IS DESIGNED WITH A MIN. 200mm TOE EMBEDMENT WITH A MIN. HORIZONTAL LEDGE WITH A GRANULAR BEDDING LAYER EXTENDING A MIN. 300mm BEYOND THE FACE AND HEEL OF THE BASE BLOCK

8. INSTALL 100mm DIAMETER PERFORATED PIPE WRAPPED WITH A GEOSOCK DRAIN BEHIND HEEL OR UNDER THE WALL. PROVIDE CLEAR STONE SURROUND TO PROTECT PIPE FROM CLOGGING AND DAMAGE. PROVIDE OUTLETS THROUGH WALL, NO FURTHER APART THAN 10.0m ON CENTRES. THE DRAINAGE PIPE SHOULD BE CONNECTED TO A POSITIVE OUTLET ON BOTH ENDS OF THE RETAINING WALL SUCH AS AN EXISTING DITCH OR CATCH BASIN.

9. THE CONDITIONS WILL BE EVALUATED BY THE GEOTECHNICAL ENGINEER DURING PREPARATION FOR WALL CONSTRUCTION IN EACH AREA. WHERE GRANULAR BEDDING WILL NOT BE SUFFICIENT THE USE OF CONCRETE BEDDING MAY BE REQUIRED.

10. ALIGNMENT OF THE BOTTOM WALL UNIT COURSE SHOULD BE PLANNED TO CONSIDER THAT A NOMINAL 41mm AUTOMATIC SETBACK WILL OCCUR WITH EACH 0.46m INCREMENT OF HEIGHT.

11. BACKFILL MATERIAL SHALL BE APPROVED BY THE SITE GEOTECHNICAL ENGINEER PRIOR TO USE AND SHOULD CONSIST OF OPSS GRANULAR B TYPE II B FOLLOWED BY SUITABLE BACKFILL MATERIAL. ALL FILL WITHIN A 1H:1V ZONE UP AND BACK FROM THE HEEL SHOULD ALSO BE COMPACTED. BACKFILL SHALL BE PLACED IN MAXIMUM 300mm LOOSE LIFTS AND COMPACTED TO A MINIMUM OF 95% OF SPMDD. MOISTURE CONTENT SHOULD BE CONTROLLED AND MAINTAINED WITHIN -3 TO +4 PERCENT OF OPTIMUM.

12. MAINTAIN TEMPORARY GRADES TO DIVERT SURFACE WATER AWAY FROM THE RETAINING WALL EXCAVATION. SLOPE FINAL BACKFILL TO PROVIDE POSITIVE DRAINAGE AND TO ELIMINATE PONDING.

13. BACKSLOPE SHOULD BE CUT BACK TO A MINIMUM OF 2H:1V TO 3H:1V TO MAINTAIN A LONG TERM SAFE SLOPE BEHIND THE RETAINING WALL. IT SHOULD BE NOTED THAT WHERE TREES ARE PRESENT WITHIN THE TOP OF SLOPE. A MINIMUM 1.0m SET BACK IS REQUIRED FOR EXCAVATION FROM THE EDGE OF THE TREE LINE WHERE PRESENT.

14. EXCAVATION SIDE SLOPES SHOULD BE PROTECTED TEMPORARILY DURING CONSTRUCTION FROM PRECIPITATION EVENTS BY PLACEMENT OF TARPS.

15. ALL RETAINING WALL RELATED INSPECTIONS (BEARING SURFACE, COMPACTION, BLOCK INSTALLATION, ETC.) MUST BE COMPLETED BY PATERSON GROUP. ONCE THE WALL CONSTRUCTION IS COMPLETED AND REVIEWED BY PATERSON DURING CONSTRUCTION, A CERTIFICATE LETTER WILL BE ISSUED BY PATERSON GROUP.

16. ANY CUTTING OF BLOCKS TO SUIT SITE CONDITIONS OR WALL DESIGN WILL BE RESPONSIBILITY OF THE CONTRACTOR.

17. IF WINTER CONSTRUCTION IS CONSIDERED, HEAT MUST BE MAINTAINED WHEN THE BASE IS EXPOSED. THE WALL BASE MUST BE COVERED WITH HIGH GRADE INSULATION TARPS TO MAINTAIN HEAT AND PROTECT THE BASE FROM POTENTIAL FROST HEAVE. ONCE THE BASE IS BACKFILLED, THE TOP OF THE WALL MUST BE COVERED WITH INSULATION TARPS OVERNIGHT UNTIL THE WALL CONSTRUCTION IS COMPLETED. ADDITION INSPECTIONS WILL BE REQUIRED DURING THE WINTER CONSTRUCTION TO ENSURE THE WALL CONSTRUCTION IS IN GENERAL CONFORMANCE WITH PATERSON'S RECOMMENDATIONS.

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL

BRIGIL

PROPOSED RETAINING WALL
TOWERS 5 & 6 - PETRIE'S LANDING

OTTAWA, ONTARIO

Title:
REDI-ROCK RETAINING WALL TYPICAL SECTION

Scale: 1:30	Date: 07/2022
Drawn by: NFRV	Report No.: PG3908-3
Checked by: JV	Dwg. No.: FIGURE 2
Approved by: FA	Revision No.:

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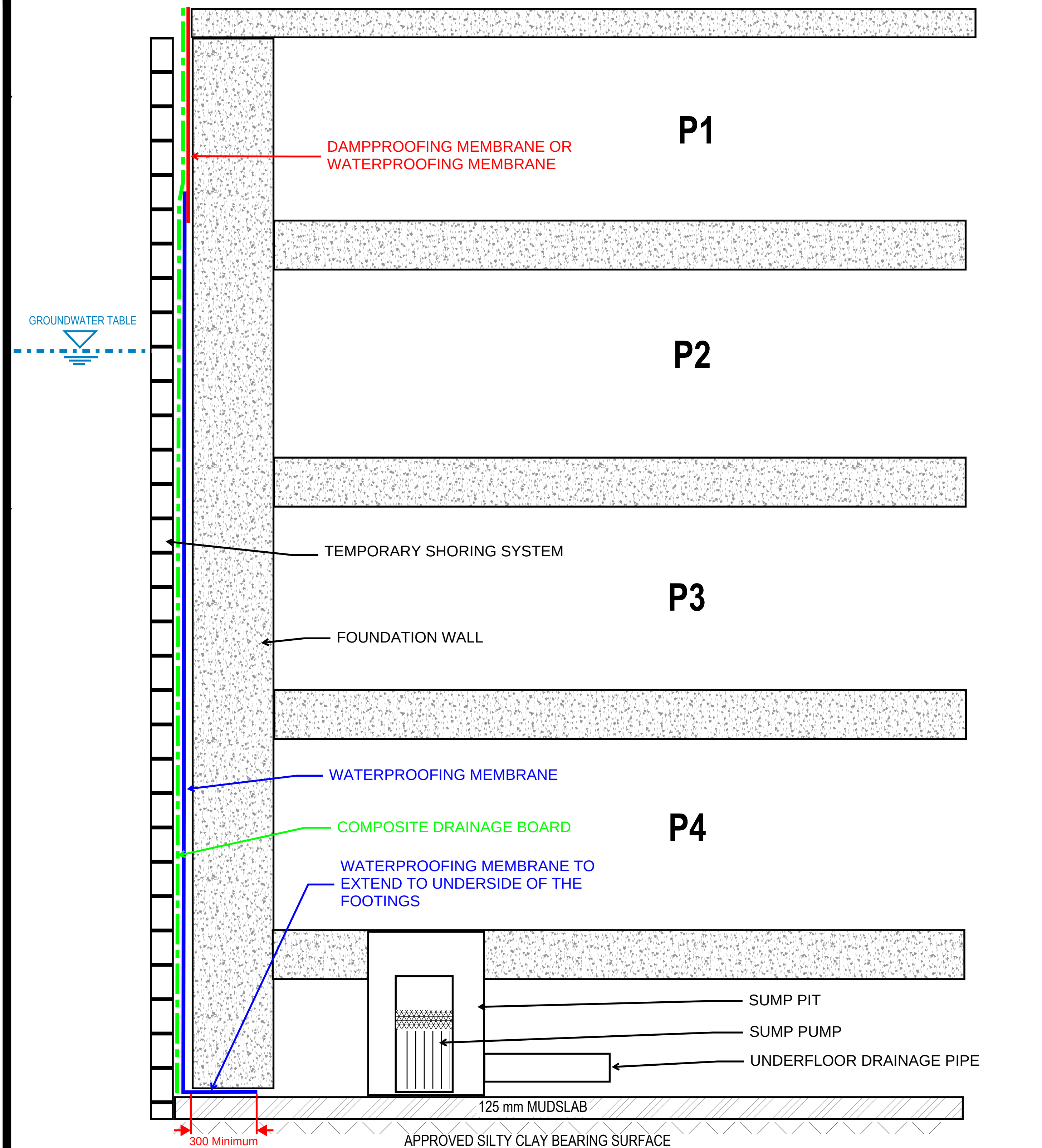

PATERSON GROUP

BRIGIL **PROPOSED MULTI-STOREY BUILDING** **PETRIE LANDING** **OTTAWA, ONTARIO**	Date: **10/2024**		Report No.: **PG3908-3**
Title: **FOUNDATION DRAINAGE SYSTEM**	Scale: **N.T.S.**		Drawing No.: **FIGURE 3**
	Drawn by: **PT**	Checked by: **JV**	Revision No.: 1

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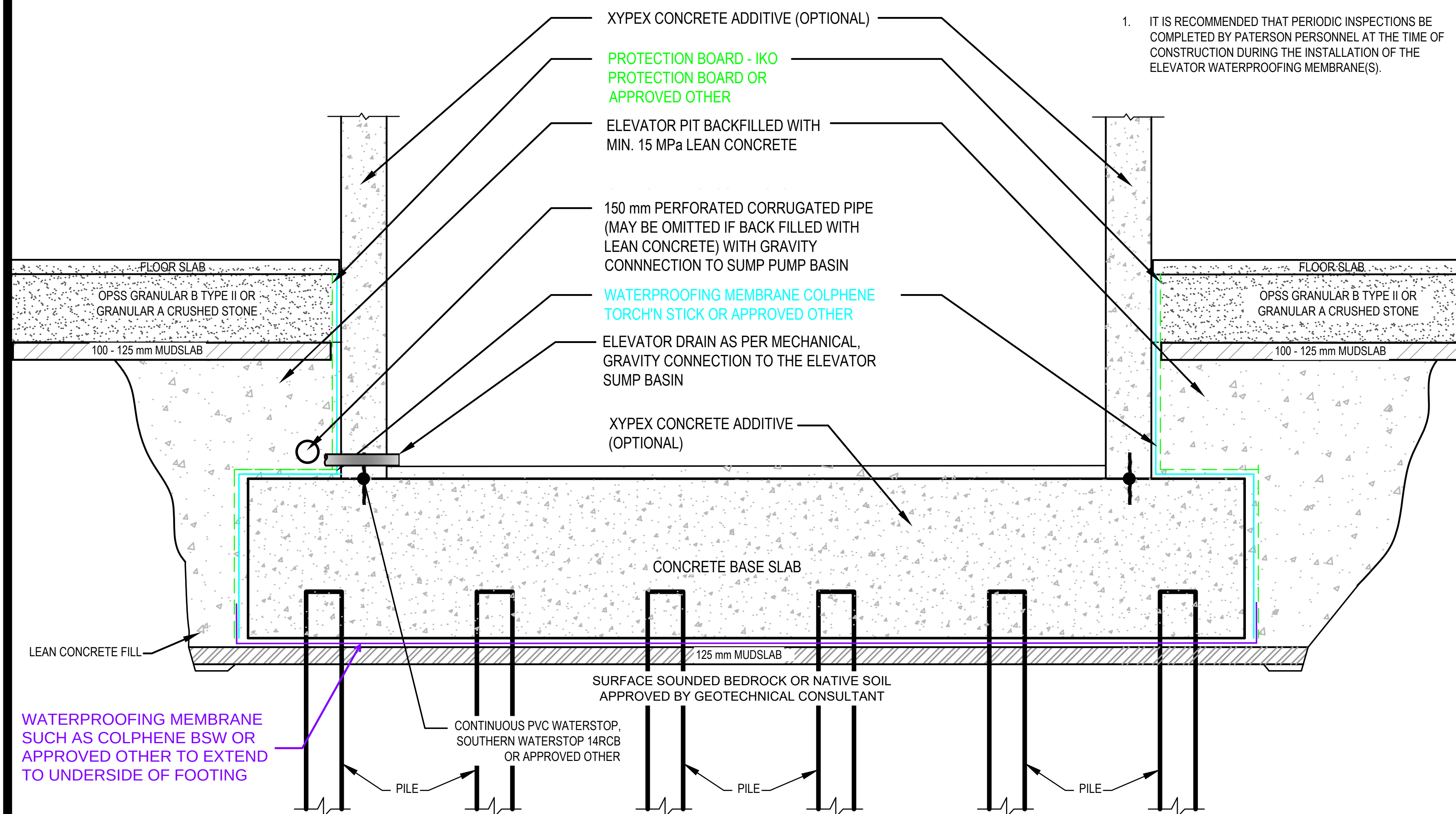

NOTES:

1. IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY PATERSON PERSONNEL AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE ELEVATOR WATERPROOFING MEMBRANE(S).

PATERSON GROUP

BIRGIL CONSTRUCTION

PROPOSED MULTI-STOREY BUILDING
TOWER 4, 5A & 5B - PETRIE'S LANDING - INLET

OTTAWA, ONTARIO

Title:

ELEVATOR PIT WATERPROOFING

Scale: N.T.S.	**Date:** 10/2024
Drawn by: PT	**Report No.:** PG3908-3
Checked by: JV	**Drawing No.:** FIGURE 4
Approved by: JV	**Revision No.:** 2

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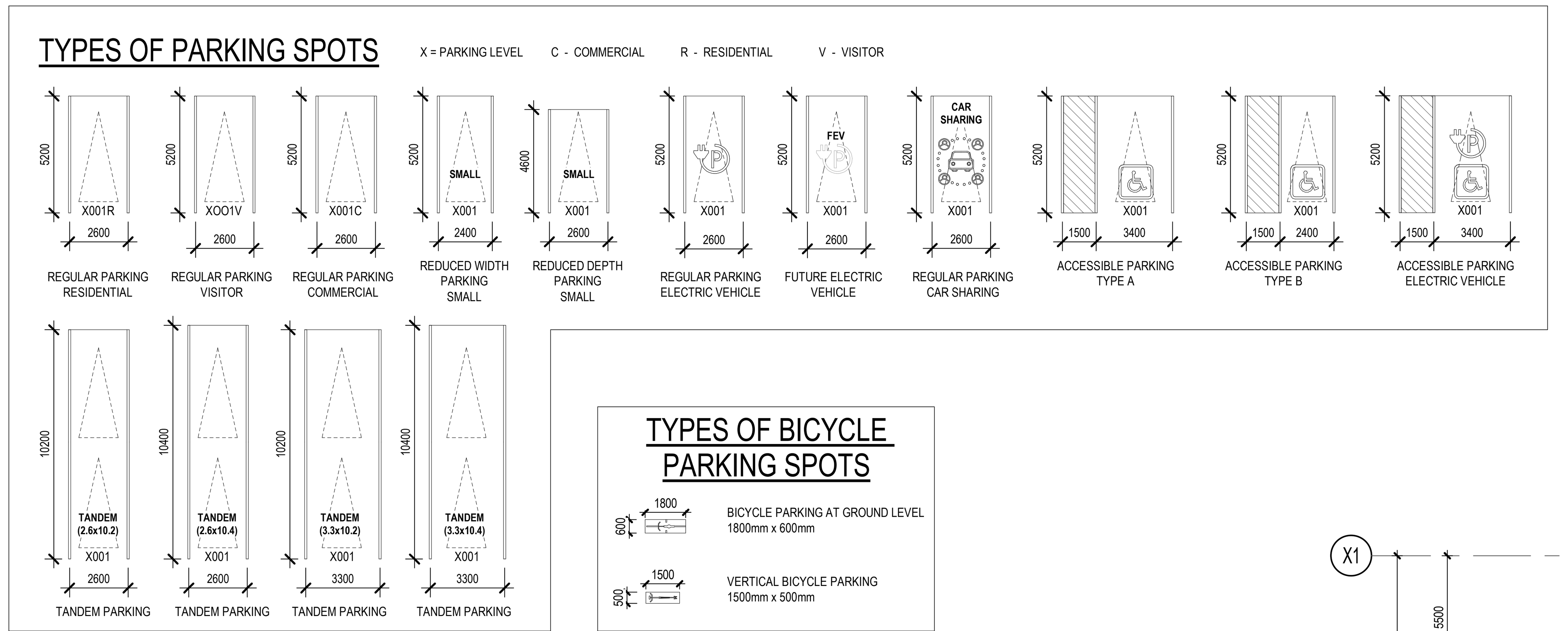
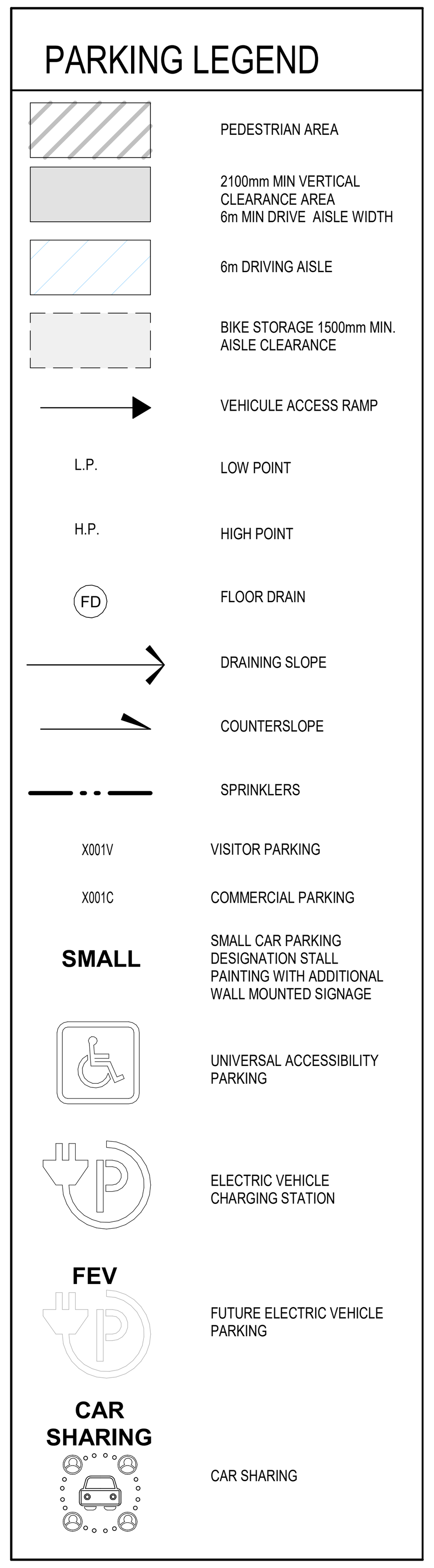


TABLEAU - ESPACE DE STATIONNEMENT	
PARKING COUNT	PARKING TYPE
BASEMENT 3	
113	2600mmx5200mm
2	2600mmx5200mm (virtucar)
7	TANDEM (2.6x10.2)
5	TANDEM (2.6x10.4)
1	TANDEM (2.6x10.4m)
1	TANDEM (2.6x10.4)
2	SMALL (2.4x5.2m)
BASEMENT 3: 131	
BASEMENT 2	
78	2600mmx5200mm
36	2600mmx5200mm (visitor)
4	2600mmx5200mm (virtucar)
2	
7	TANDEM (2.6x10.2)
1	TANDEM (2.6x10.4)
1	TANDEM (2.6x10.4m)
1	TANDEM (2.6x10.4)
2	SMALL (2.4x5.2m)
BASEMENT 2: 132	
BASEMENT 1	
40	2600mmx5200mm (commercial)
63	2600mmx5200mm (visitor)
2	SMALL (2.4x5.2m)
2	3400mmx5200mm plus aisle HANDICAP (Commercial)
7	3400mmx5200mm plus aisle HANDICAP (Visitor)
5	3660mmx5200mm HANDICAP (Visitor)
BASEMENT 1: 119	
Total général: 382	



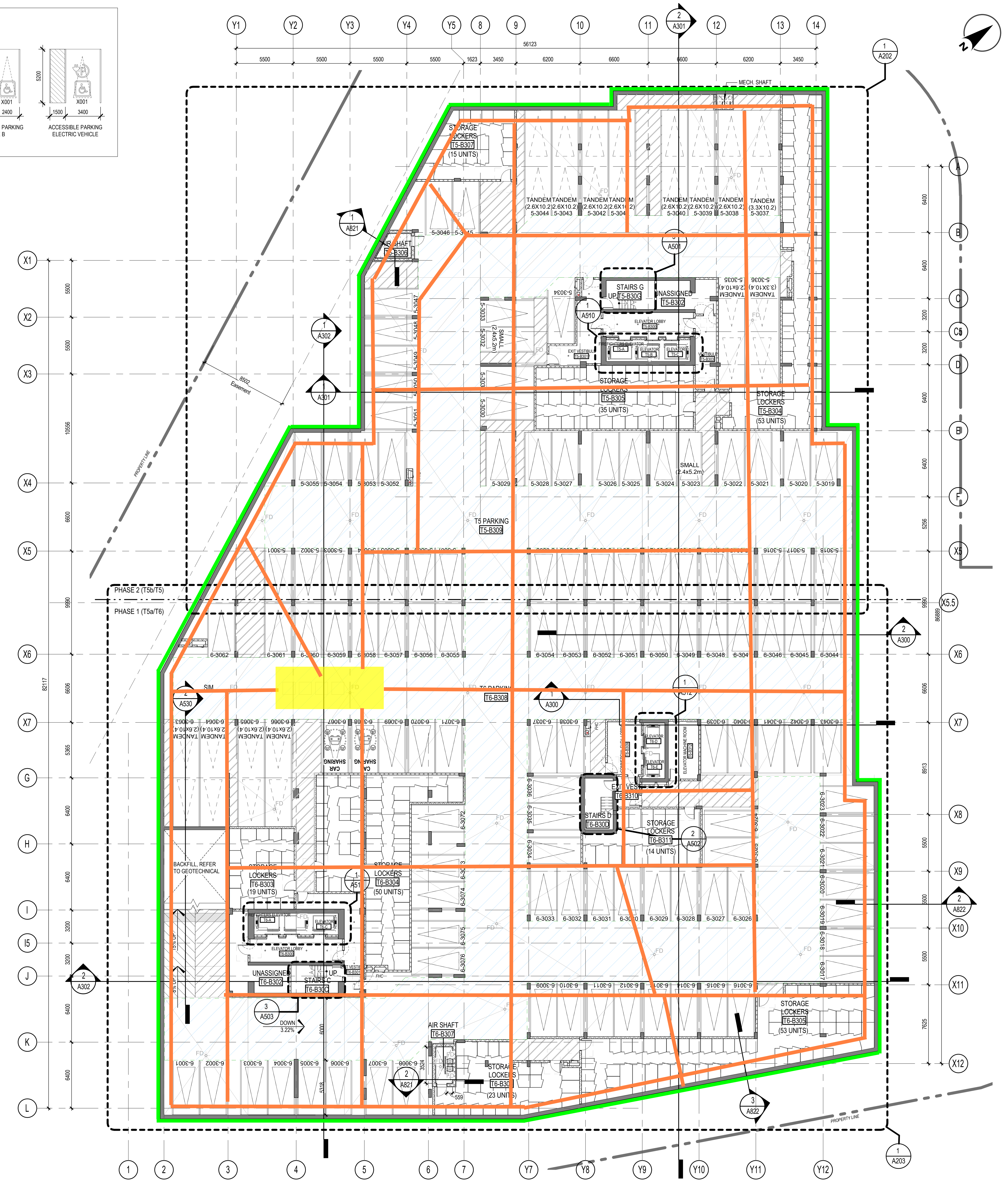


Figure 5 - Subfloor Drainage Layout

Waterproofing Assembly as per Figure 3

Min 150 mm perforated drainage pipe placed below slab and between pile caps to promote drainage to sump pit

Approximate sump pit area (to be coordinated with mechanical)

NOTES GÉNÉRALES General Notes

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STRUCTURAL Structure
Leroux & Cyr
130 blvd. Henri-Bourassa Est, Montréal QC H3L 1B7
T: 438-381-7773 Email: gleroux@lerouxcyr.com

MECANICAL/ELECTRICAL Mécanique/Électrique
Goodkey, Weedmark & Associates Ltd.
1688 Woodward Drive, Ottawa ON K2C 3R8
T: 613-727-5111 Email: m.sarasin@gwal.com / rajv@gwal.com

LANDSCAPE ARCHITECT Architect paysager
Levstek Consultants Inc.
5871 Hugh Crescent, Ottawa ON K0A 2W0
T: 613-826-0518 (203) Email: rlevstek@larocquelevstek.com

GEOTECHNICAL Géotechnique
Paterson Group
9 Auriga Drive, Ottawa ON K2E 7T9
T: 613-226-7381 Email: jvilleneuve@patersongroup.ca

CIVIL Civil
exp Services Inc.
2650 Queensview Drive, Suite 100, Ottawa ON K2B 8H6
T: 613-688-1899 Email: Chris.collins@exp.com

ARCHITECTES Architect
NEUF architect(e)s
630, boul. René-Lévesque O. 32e étages, Montréal QC H3B 1S6
T 514 847 1117 NEUFarchitectes.com

SCEAU / Seal

NEUF ARCHITECT(E)S

NEUF ARCHITECTES INC.

CLIENT Client

brigil

OUVRAGE Project

PETRIE'S LANDING I - TOWER 5/6

EMPLACEMENT Location: ORLEANS, ON

NO PROJET No.: 11597/12191

NO	RÉVISION	DATE (aa-mm-jj)
1	ISSUED FOR INTERNAL REVIEW - ARCHITECT	2021-10-18
A	ISSUED FOR 30%	2021-10-29
B	ISSUED FOR SPA REV6	2021-11-02
C	ISSUED FOR 60%	2021-12-16
D	RE-ISSUED FOR SPA REV6	2022-02-28
E	ISSUED FOR 80% REVIEW	2022-04-21
F	ISSUED FOR PERMIT	2022-07-15
G	ISSUED FOR TENDER	2022-12-02
H	RE-ISSUED FOR TENDER	2023-03-10
I	ISSUED FOR 60% COORD.	2024-03-22

DESSINÉ PAR Drawn by: AJ

VÉRIFIÉ PAR Checked: MU

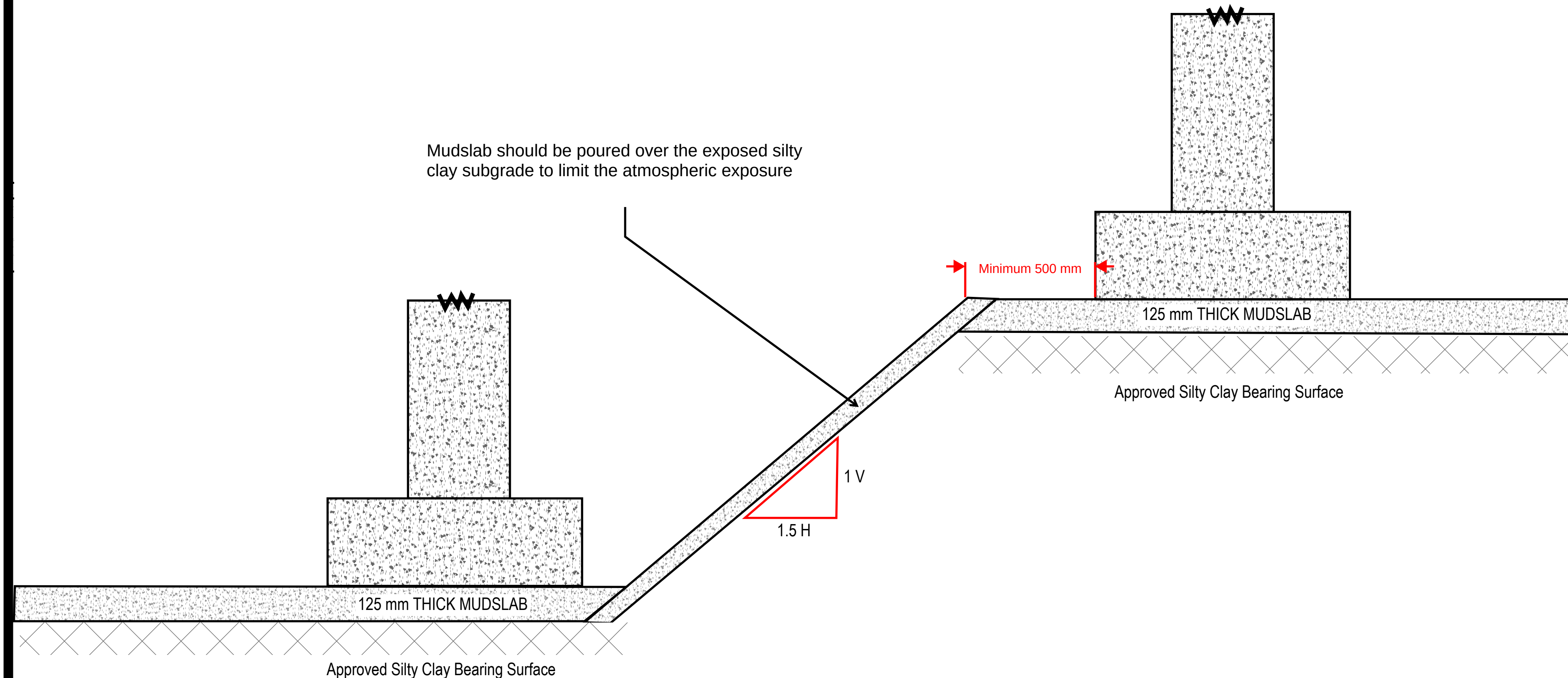
DATE (aa.mm.jj): 10/29/21

ÉCHELLE Scale: As indicated

TITRE DU DESSIN Drawing Title

RÉVISION Revision: I

NO. DESSIN Dwg Number: A201



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

BRIGIL
PROPOSED MULTI STOREY BUILDINGS
PETRIE LANDING

OTTAWA, ONTARIO

Title:

MUDSLAB DETAIL

Scale: N.T.S.	**Date:** 12/2024
Drawn by: PT	**Report No.:** PG3908-3
Checked by: PT	**Drawing No.:** FIGURE 6
Approved by: JV	**Revision No.:**

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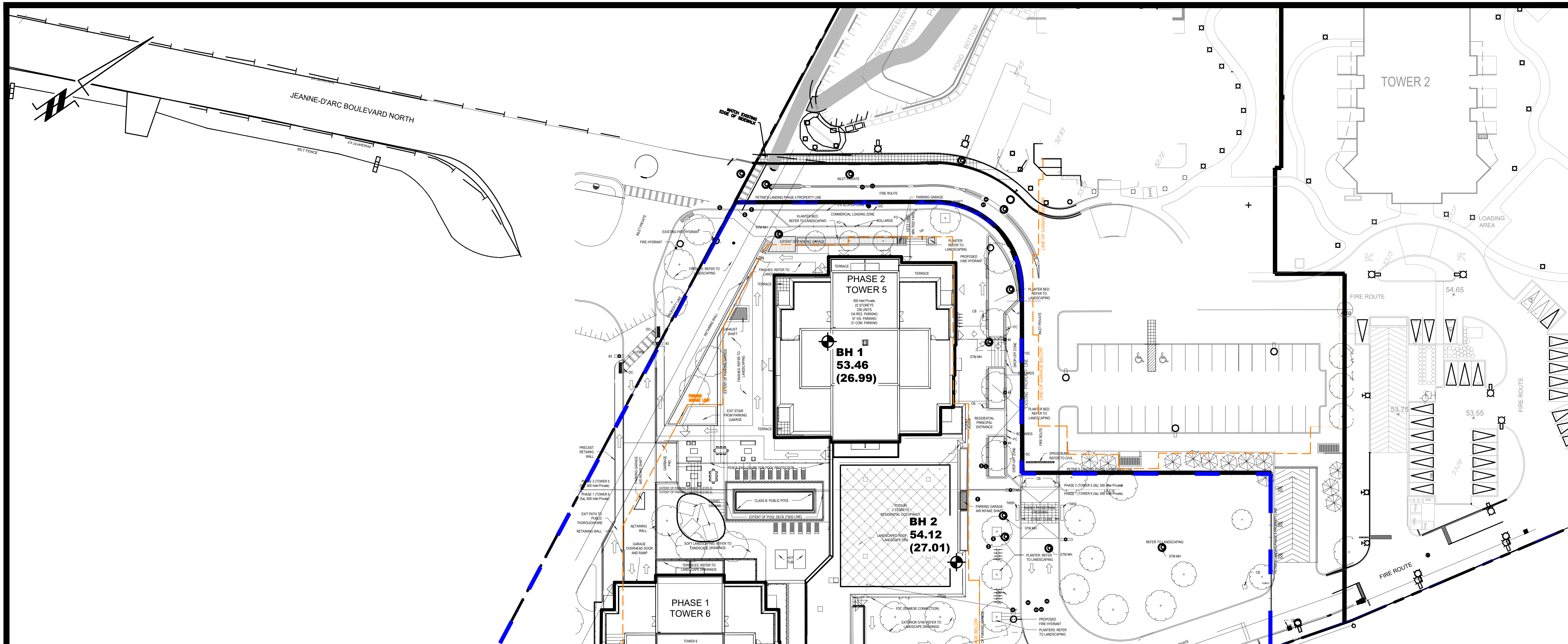

LEGEND:

BOREHOLE LOCATION

53.46 GROUND SURFACE ELEVATION (m)

(26.99) PRACTICAL DCPT REFUSAL ELEV. (m)

CONCEPTUAL PLAN NEUF ARCHITECT(E)S.

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

SCALE: 1:750

0 10 20 30 50m

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7S9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
4	UPDATED CONCEPTUAL PLAN	27/11/2025	PT
3	UPDATED CONCEPTUAL PLAN	07/11/2025	PT
2	UPDATED CONCEPTUAL PLAN	04/11/2024	PT
1	REVISED AS PER CITY COMMENTS	18/04/2024	FV

BRIGIL CONSTRUCTION

GEOTECHNICAL INVESTIGATION

PROPOSED HI-RISE BUILDINGS - TOWERS 5 & 6 - PETRIE'S LANDING

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale:	1:750	Date:	07/2022
Drawn by:	NFRV	Report No.:	PG3908-3
Checked by:	BN	Dwg. No.:	PG3908-1
Approved by:	JV	Revision No.:	4

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