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Geotechnical Investigation

Proposed Hi-Rise Buildings
Towers 3, 4, 5A and 5B - Petrie's Landing
8900 Jeanne D'Arc Boulevard
Ottawa, Ontario

Prepared For

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by 6382983 Canada Inc. to carry out a geotechnical investigation for the proposed high-rise buildings (Tower 3, 4, 5A and 5B) 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 complex will include the development of 4 residential towers (Towers 3, 4, 5A and 5B) consisting of 18, 22, 32 and 22 storeys, respectively. It is understood that 2 levels of underground parking are proposed for Tower 3 while Towers 4, 5A and 5B will include 3 levels of underground parking. 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. 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 PG3908-1 - 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.

Sample Storage

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

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 PG3908-1 - 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.

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 is occupied by agricultural fields and is relatively flat. An approximately 2 m high fill pile was observed within the footprint of the proposed Tower 3. 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 south and southwest by an existing residential high-rise building (Tower 2) and the associated above ground parking areas. Also, a gravel covered access road was noted and located within the footprint of Tower 5A and 5B.

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 32 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 3, 4, 5A and 5B). 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 will be founded on conventional spread footings 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.

Protection of Existing Footings

Due to the close proximity of Tower 2 with respect to the proposed Tower 3, as well as tower 4 following the construction of Tower 3, it is recommended that the lateral support zone of the existing footings be protected. Therefore, the excavation is recommended to be at a minimum of 1H:1V of the outer edge of footing of the footings of the existing building.

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

5.2 Site Grading and Preparation

Stripping Depth

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

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.

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 40.0 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.

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.

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 Design

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

For Tower 3, a perimeter foundation drainage system is recommended to be provided for the proposed building. The system should consist of a combination of a composite drainage system and a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer. Refer to Figure 3 in Appendix 2.

Water Suppression System

For Towers 4, 5A and 5B, a water suppression system will be required for the lower P-3 Level to avoid dewatering the surrounding areas adjacent to buildings with shallower founding depths (P-2 Level) which can cause differential settlement. To manage and control groundwater water infiltration over the long term, the following water suppression system is recommended to be installed for the exterior foundation walls and underfloor drainage (refer to Figure 2 - Water Suppression System in Appendix 2 for an illustration of this system cross-section):

☐ A concrete mud slab will be required to create a horizontal hydraulic barrier to lessen the water infiltration at the base of the excavation and will consist of a 300 mm thick layer of 25 MPa compressive strength concrete. The 300 mm minimum thickness is required to enable the support of construction traffic until the footings, pile caps and grade beams are poured and the area is backfilled for the lower floor slab to resist minor buoyancy forces and hydrostatic pressure.

☐ A waterproofing membrane will be required to lessen the effect of water infiltration for the underground parking P-3 Levels starting at underside of P-2 Level which is approximately 7 m below finished grade (which is approximately 1 m above the expected high groundwater table). Since space is available for an open cut excavation, the waterproofing system will consist of a membrane layer capable of being fastened and sealed against a composite drainage layer and concrete foundation wall. The membrane should extend to the bottom of the excavation at the founding level of the proposed footings over the concrete mud slab.

- ☐ A composite drainage layer will be placed from finished grade to the bottom of the foundation wall. It's recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the bottom of the foundation wall. It's expected that 150 mm diameter sleeves placed at 3 m centres be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to the sump pit(s) within the lower basement area. Water infiltration will result from two sources. The first will be water infiltration from the upper 7 m which is above the vertical waterproofed area. The second source will be groundwater breaching the waterproofing membrane.

Construction details for installing the membrane, drainage board and mudslab are presented in Figure 2 to 8 in appendix 2.Membranes and drainage board should be installed as per manufacturer's specification. Paterson should review any proposal by supplier prior to the field work.

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 150 mm thick concrete mud slab). For design purposes, it's recommended that a 150 mm diameter perforated pipe be placed in each bay over the concrete hydraulic barrier. For preliminary purposes the pipe spacing should range from 6 to 8 m. The final spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed. Reference should be made to Figure 11 in Appendix 2.

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 elevator 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 mudslab in accordance with the manufacturers 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.

The foundation wall of the elevator shaft and buildings sump pit should host a PVC sleeve to allow any water trapped within the interior side of the structures to be discharged to the associated sump pump. A minimum 100 mm diameter perforated, corrugated drainage pipe should extend from the sleeve towards the associated drainage system by gravity drainage and mechanical connection to the associated system. Also, the contractor should ensure that the opening is properly sealed to prevent water from entering the subject structure.

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

It should be noted that a waterproofed concrete (with Xypex Additive, or equivalent) is optional for this waterproofing option. Refer to Figure 9- Elevator and Sump Pump Waterproofing Detail, for specific details of the waterproofing recommendation attached to the current memorandum.

Water Infiltration Volumes

During the construction phase, it's expected that water infiltration should have a steady state volume of less than 50,000 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 3 levels of underground parking will be approximately 10 m.

With the water suppression system in place, it's expected that long term groundwater infiltration will be significantly reduced during post-construction. With a properly implemented water suppression system, it's expected that post-construction volumes will be less than 10,000 L/day. The zone of influence associated with the long term dewatering at post construction for 3 levels of underground parking will be less than 5 m.

The proposed water suppression system will control the water influx and all water breaching the system of from surface infiltration should be pumped by the building's sump pit(s).

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, connected to the perimeter foundation drainage system. 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 10 m from the foundation perimeter and less than 5 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 water suppression 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 4 will be located below the 100 year flood level. To limit long-term groundwater infiltration, it's recommended that a flood proofing system be designed for the proposed building. The recommended water suppression system will also be used as the flood proofing system to lessen the infiltration volumes and manage discharge. Also, an interior perimeter foundation drainage system will be required as a secondary system to account for any groundwater which breaches the primary groundwater infiltration control system.

It is important to note that the building's sump pit and elevator pit be waterproofed. A detail can be provided by Paterson once the design drawings are available for the elevator and sump pits.

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 a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

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

Temporary Shoring

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

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

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

Table 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 are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

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

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & 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, subbedding 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 stratigic 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.

It is also proposed to use a pressure relief chamber to control groundwater infiltration. All construction time dewatering can be completed thought the pressure relief chamber described below. For short term and construction purposes no pumping limit is applicable from a geotehcnical perspective. It is expected that pumping will be stopped when sufficient dead load has been applied to the structure and the water suppression system can take over.

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

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

Pressure Relief Chamber

A pressure relief chamber must be installed to manage construction and long-term dewatering of groundwater surrounding the site, a pressure relief chamber will be installed along with collection pipes within the silty clay deposit. The collection pipe trenching should extend along the proposed building perimeter and lead to the pressure relief chamber. It is suggested that the pressure relief chamber be incorporated in the lowest section of the P3 level within a utility room in close proximity to the proposed sump pit(s). Figure 7 - Pressure Relief Chamber in Appendix 2 provides the requirements for the pressure relief chamber. Once the pressure relief chamber and associated piping is installed, the proposed raft slab can be constructed. The purpose of the pressure relief chamber will be as follows:

- ❏ Decrease basal heave potential by redirecting depressurized groundwater to the pressure relief chamber(s).

- ❏ Manage any water infiltration along the founding surface during the excavation program.
- ❏ Manage the water infiltration during the pouring of the concrete hydraulic barrier slab to prevent water flow in the fresh concrete.
- ❏ Manage water infiltration below the floor slab until sufficient load is applied to resist any potential hydrostatic uplift.
- ❏ Regulate the discharge valve to control water infiltration once the parking garage is in place and over the long term to manage the hydrostatic pressure to permit any repairs associated with any water infiltration.
- ❏ Once sufficient load is applied to the floor slab, the pressure relief valve will be fully closed to prevent any further dewatering.
- ❏ A digital monitoring pressure gauge should be installed to maintain the hydrostatic pressures as indicated in the following section.

Hydrostatic Pressure

With the fully closed valve within the pressure relief chamber and a perfectly watertight foundation, it is expected that a maximum hydrostatic pressure of 30 kPa will be developed over the long term and should be incorporated in the design of the foundation and the foundation wall. However, since this will be a water suppression system, minor water infiltration will be managed and will significantly reduce any potential for hydrostatic pressure build up. Therefore, a realistic long term hydrostatic pressure will be closer to 10 or 15 kPa which is essentially the loading applied by the lower level floor slab, granular and concrete mud slab.

The purpose of the pressure relief chamber will be to control the groundwater infiltration and hydrostatic pressure created by a partially tanking the basement level. To avoid uplift on the P3 Level floor slab prior to having sufficient loading to resist uplift, it's recommended that the water infiltration be pumped via the pressure relief chamber during the construction program.

During the construction program, the valve of the pressure relief chamber can be gradually closed as the loading is applied to resist hydrostatic pressure. Once sufficient load is available to resist the full hydrostatic pressure, the valve of the pressure relief chamber can be adjusted and closed to minimize water infiltration volumes.

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 perimeter or 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 10,000 L/day per building) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. The groundwater flow should be controllable using conventional open sumps.

The pressure relief chamber should also be connected to the sump pump. An automatic valve can be install to automatically discharge pressure into the sump pits. The automation should be set for a pressure exceeding 30 kPa.

Impacts on Neighbouring 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 neighbouring 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 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.

6.7 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 materials 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. Also, our recommendations should be reviewed when the project drawings and specifications are complete.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request that we be notified immediately to permit reassessment of our recommendations.

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

Paterson Group Inc.

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

David J. Gilbert, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Nov 26, 2021
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Report Distribution

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

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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

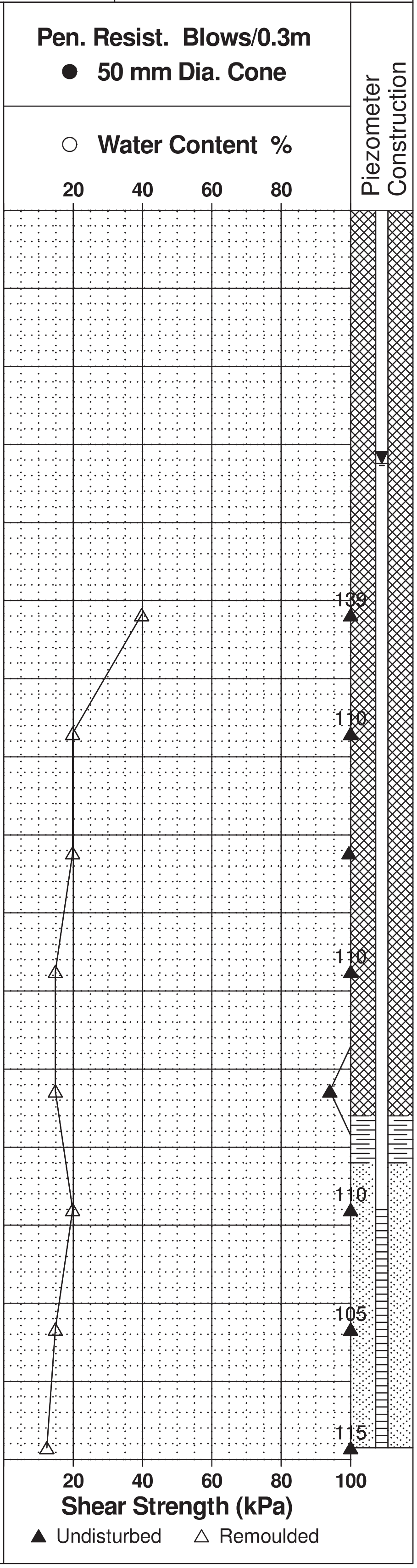
BORINGS BY CME 55 Power Auger

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							
FILL: Brown silty sand with crushed stone 0.53		AU	1			0	53.46
						1	52.46
						2	51.46
						3	50.46
						4	49.46
Very stiff, brown **SILTY CLAY**						5	48.46
- grey by 5.5m depth						6	47.46
						7	46.46
						8	45.46
						9	44.46
						10	43.46
						11	42.46
						12	41.46
						13	40.46
						14	39.46
						15	38.46
15.85						16	37.46



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Geotechnical Investigation
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DATE July 16, 2018

FILE NO. PG3908

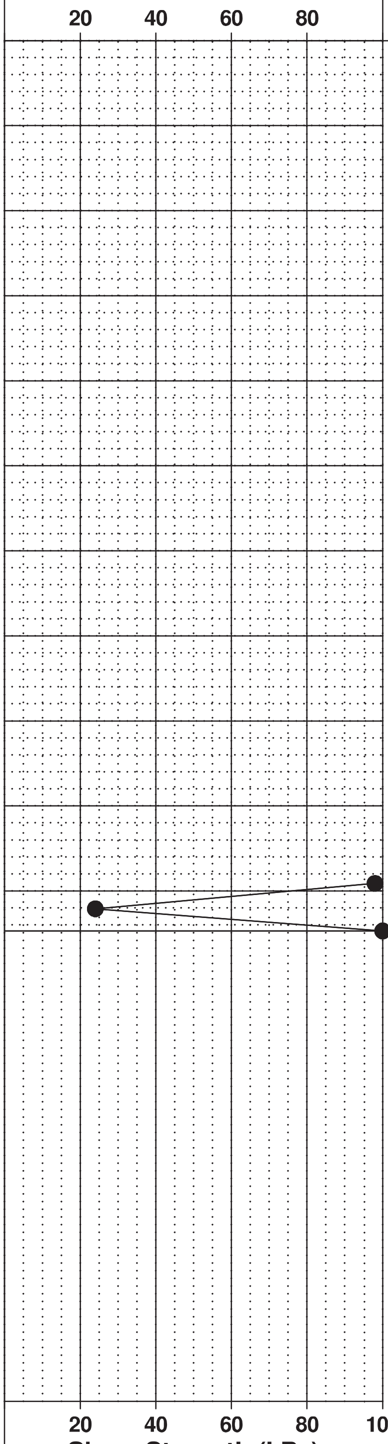
HOLE NO. BH 1

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	27.46
26.47							
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 %

Piezometer Construction



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

SOIL PROFILE AND TEST DATA

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Proposed Development - Petrie's Landing I
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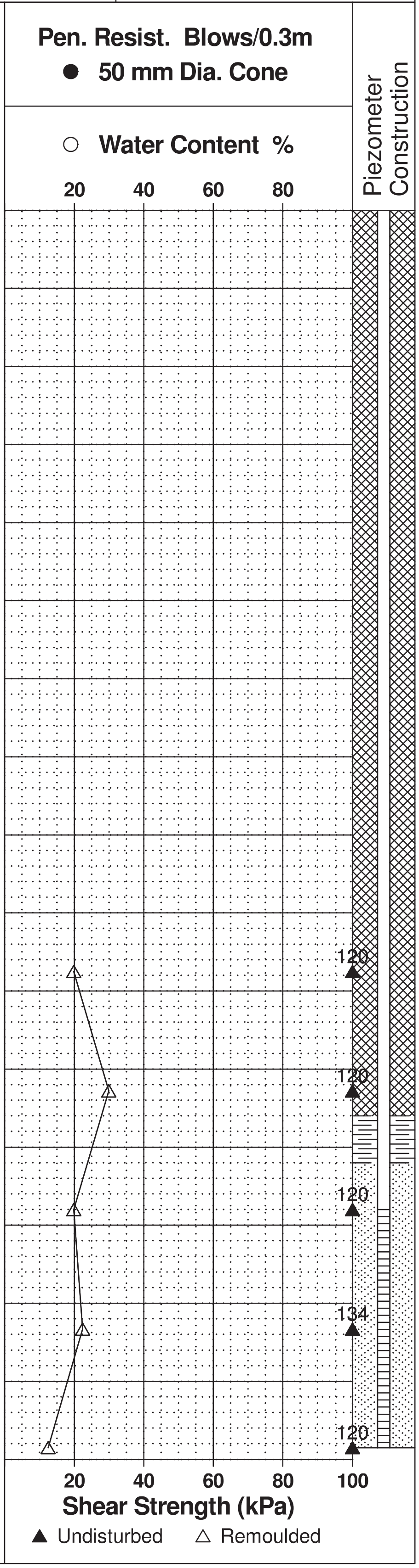
DATE July 13, 2018
FILE NO. PG3908
HOLE NO. BH 2

SOIL DESCRIPTION	Layer bottom depth (m)	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE						
FILL: Brown silty sand with crushed stone and organics	0.48	▨	AU	1		
Very stiff, brown **SILTY CLAY** - grey by 6.3m depth	15.85	▨				

DEPTH (m)	ELEV. (m)
0	54.12
1	53.12
2	52.12
3	51.12
4	50.12
5	49.12
6	48.12
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
16	38.12



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

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REMARKS

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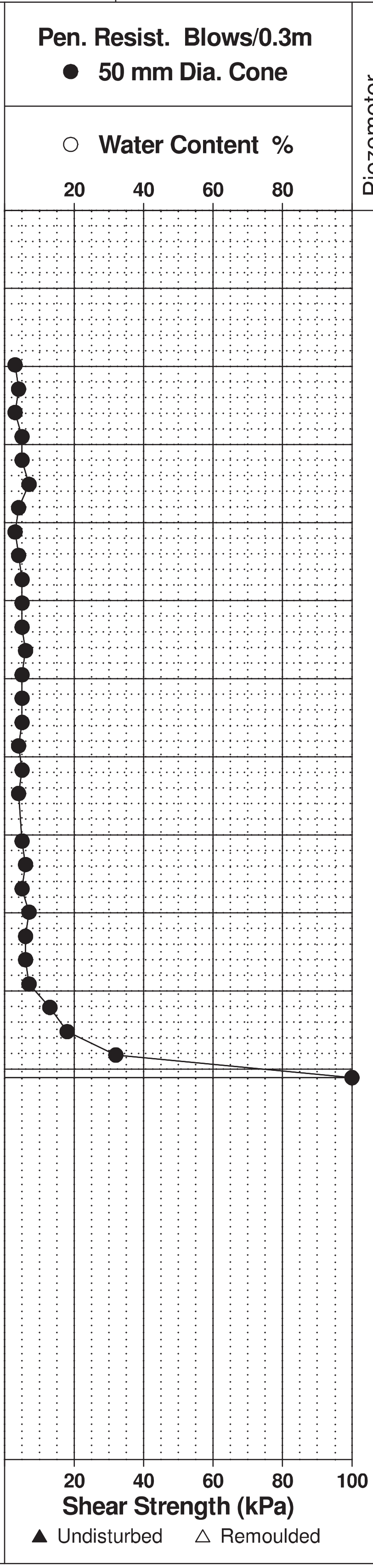
DATE July 13, 2018

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)							



Piezometer Construction

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SOIL PROFILE AND TEST DATA

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

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REMARKS

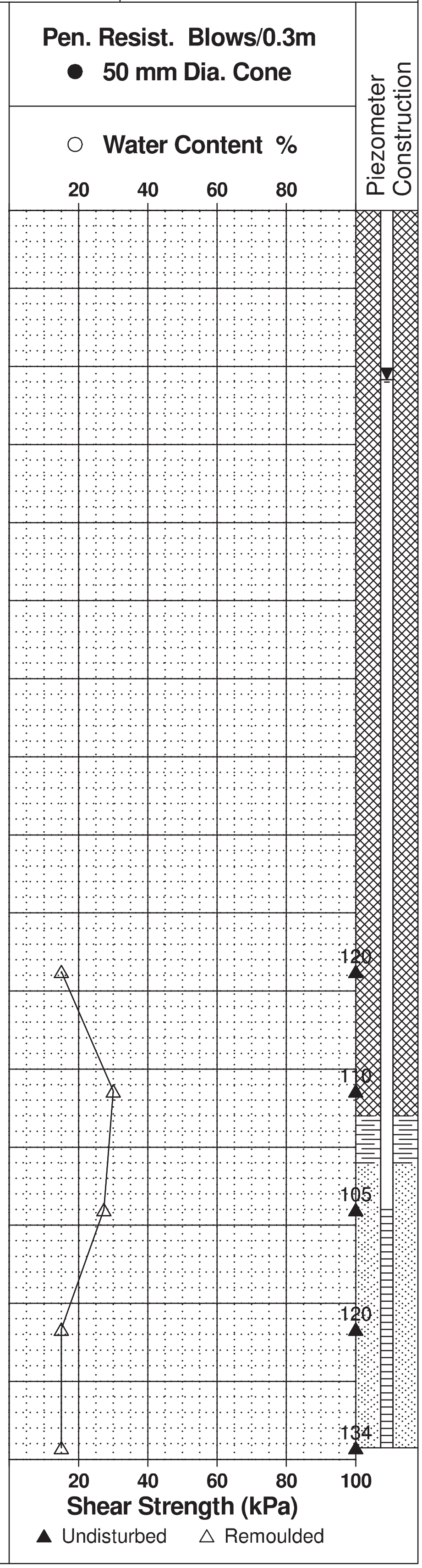
BORINGS BY CME 55 Power Auger

DATE July 16, 2018

FILE NO. PG3908

HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						
TOPSOIL 0.05	AU	1			0	53.42
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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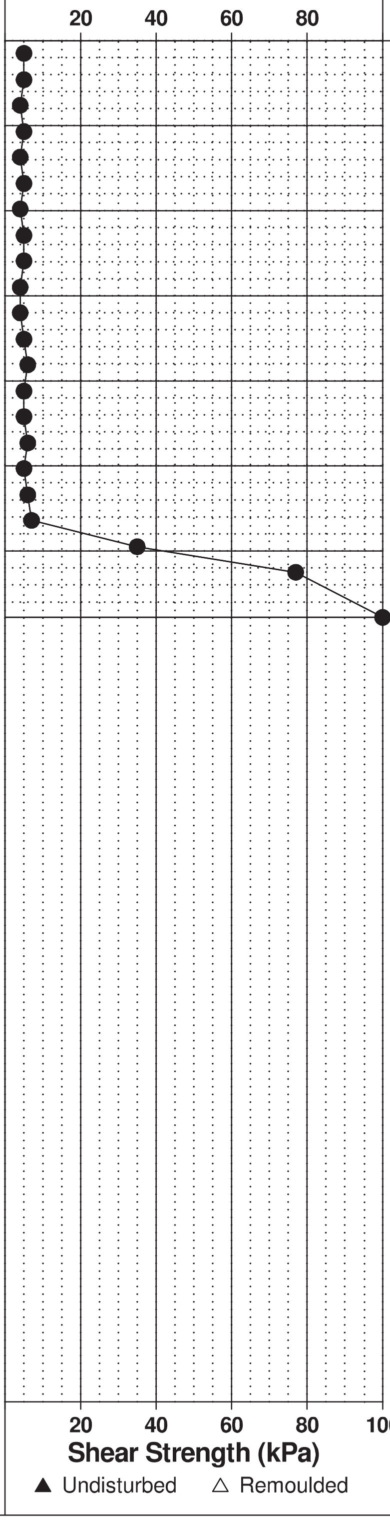
DATE July 16, 2018

FILE NO. PG3908

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
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	31.42		
22.78									
End of Borehole Practical DCPT refusal at 22.78m depth. (GWL @ 2.17m - July 24, 2018)									

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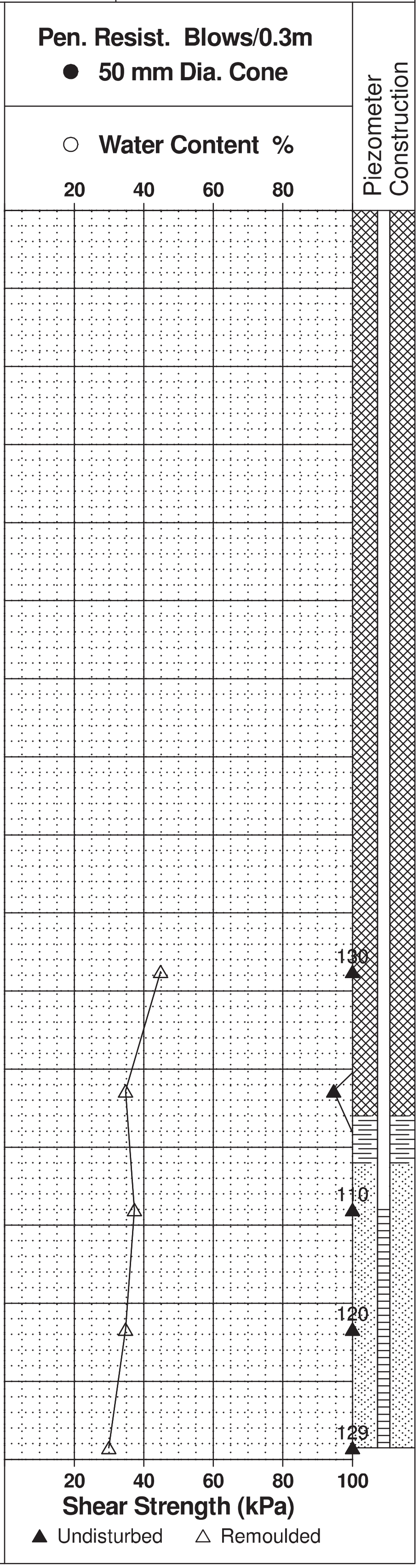
FILE NO. PG3908

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
FILL: Brown silty clay with crushed stone and organics 0.18		AU	1			0	53.26
						1	52.26
						2	51.26
						3	50.26
						4	49.26
						5	48.26
						6	47.26
Very stiff, brown **SILTY CLAY**						7	46.26
- grey by 7.6m depth						8	45.26
						9	44.26
						10	43.26
						11	42.26
						12	41.26
						13	40.26
						14	39.26
						15	38.26
15.85						16	37.26



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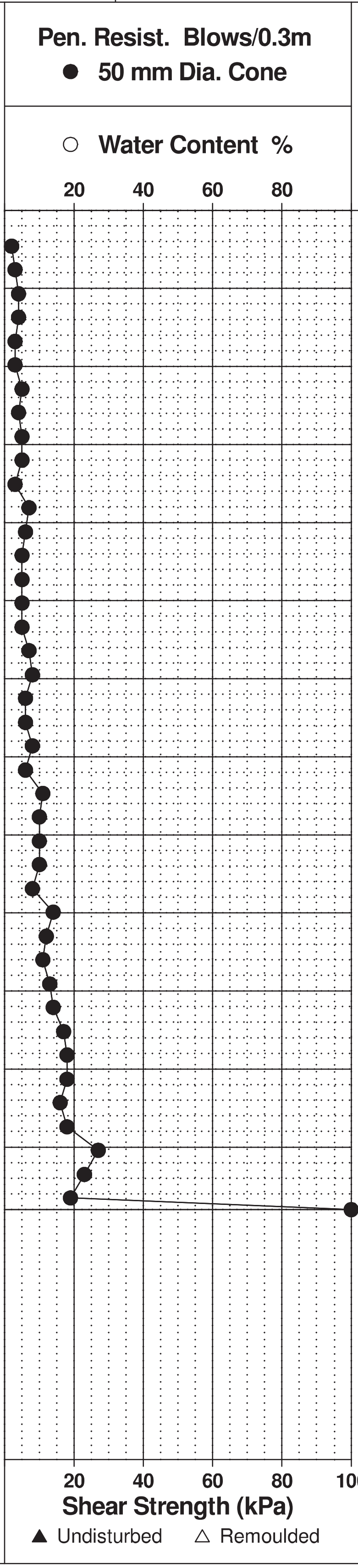
HOLE NO. BH 4

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.						16	37.26
						17	36.26
						18	35.26
						19	34.26
						20	33.26
						21	32.26
						22	31.26
						23	30.26
						24	29.26
						25	28.26
						26	27.26
						27	26.26
						28	25.26
28.80							
End of Borehole Practical DCPT refusal at 28.80m depth. (BH dry - July 24, 2018)							



Piezometer Construction

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SOIL PROFILE AND TEST DATA

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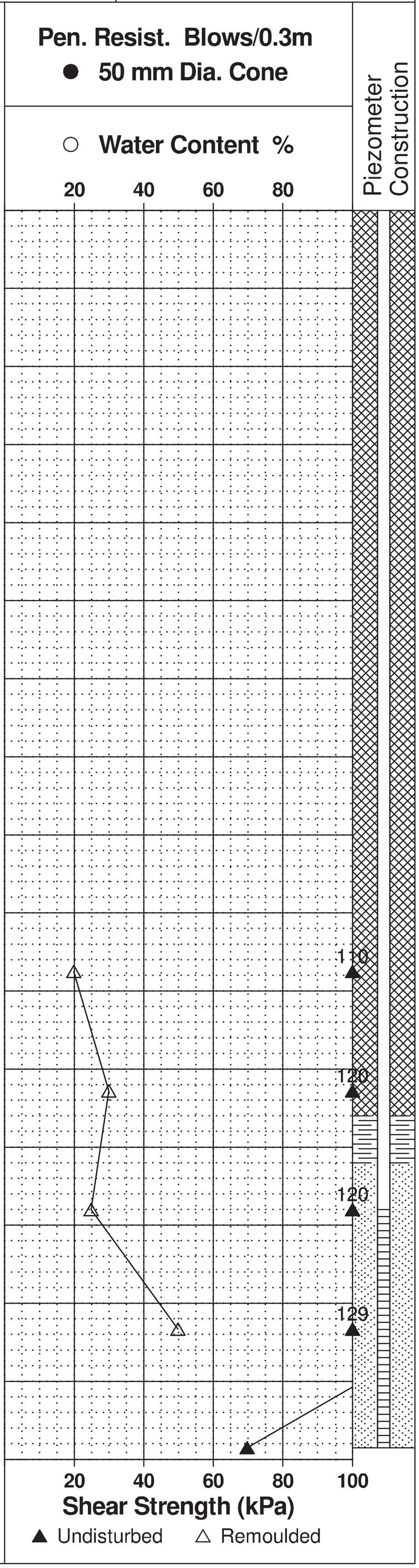
DATE July 17, 2018

FILE NO. PG3908

HOLE NO. BH 5

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty clay, trace gravel and organics 0.10	AU	1		
Very stiff, brown **SILTY CLAY** - grey by 6.8m depth				
15.85				

DEPTH (m)	ELEV. (m)
0	52.78
1	51.78
2	50.78
3	49.78
4	48.78
5	47.78
6	46.78
7	45.78
8	44.78
9	43.78
10	42.78
11	41.78
12	40.78
13	39.78
14	38.78
15	37.78
16	36.78



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SOIL PROFILE AND TEST DATA

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100 Inlet Private, Ottawa, Ontario

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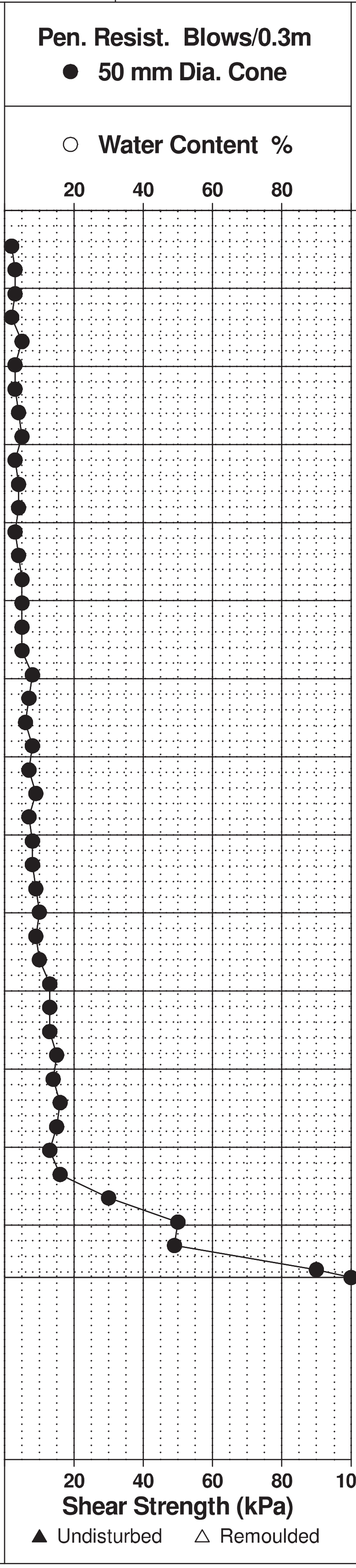
REMARKS

HOLE NO. BH 5

BORINGS BY CME 55 Power Auger

DATE July 17, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE								
Dynamic Cone Penetration Test commenced at 15.85m depth.						16	36.78	
						17	35.78	
						18	34.78	
						19	33.78	
						20	32.78	
						21	31.78	
						22	30.78	
						23	29.78	
						24	28.78	
						25	27.78	
						26	26.78	
						27	25.78	
						28	24.78	
						29	23.78	
29.67								
End of Borehole Practical DCPT refusal at 29.67m depth. (BH dry - July 24, 2018)								



SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

FILE NO. PG3908

REMARKS

HOLE NO. BH 6

BORINGS BY CME 55 Power Auger

DATE July 18, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with crushed stone and organics (to 0.46)		AU	1		
Very stiff, brown **SILTY CLAY** - grey by 6.4m depth (to 15.85)					

DEPTH (m)	ELEV. (m)
0	53.09
1	52.09
2	51.09
3	50.09
4	49.09
5	48.09
6	47.09
7	46.09
8	45.09
9	44.09
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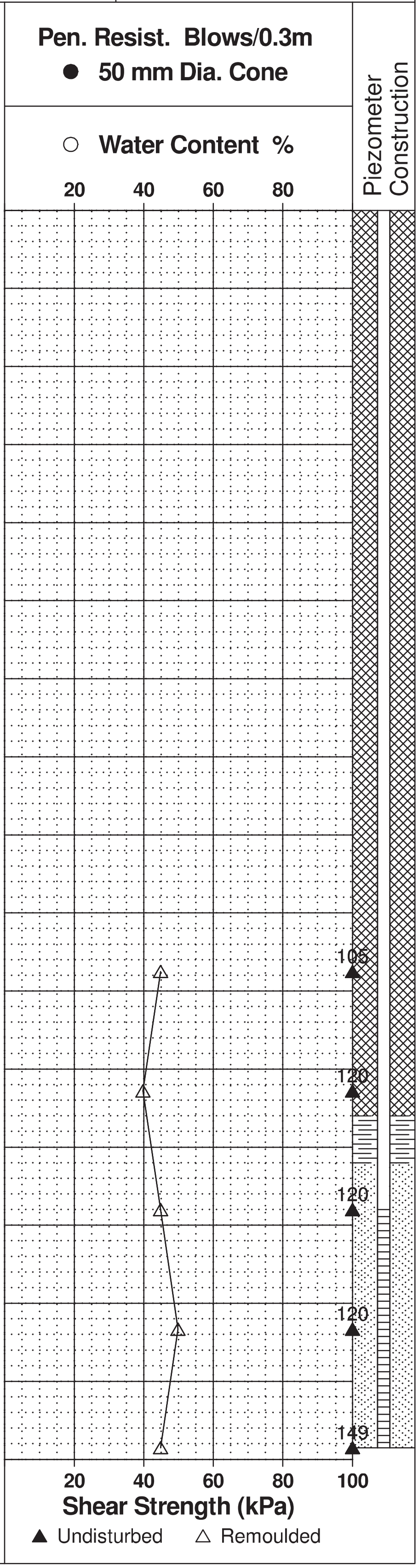
15	38.09
16	37.09



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

DATUM Geodetic

REMARKS

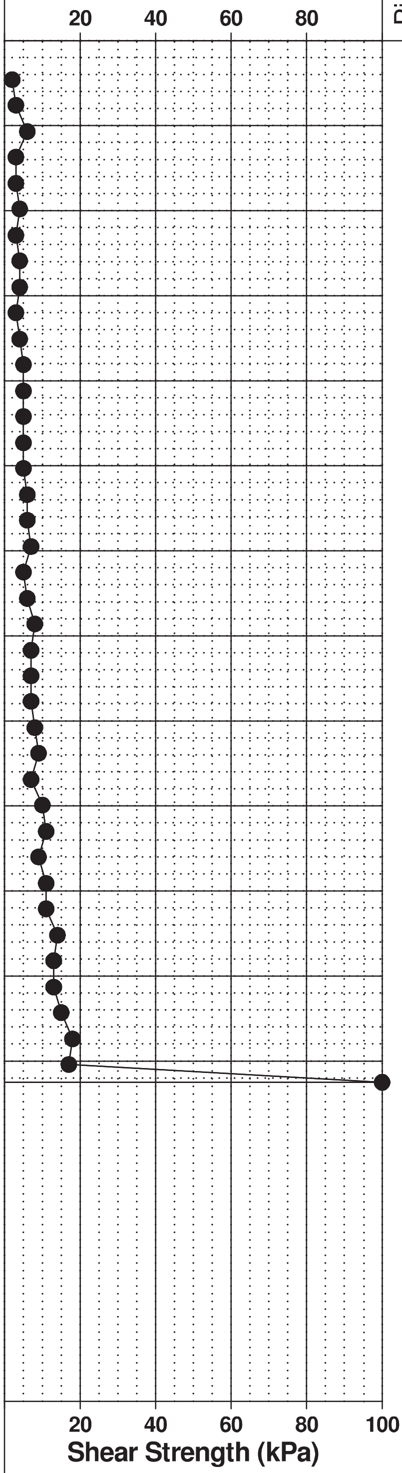
BORINGS BY CME 55 Power Auger

DATE July 18, 2018

FILE NO. PG3908

HOLE NO. BH 6

SOIL DESCRIPTION GROUND SURFACE	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
Dynamic Cone Penetration Test commenced at 15.85m depth.						16	37.09		
						17	36.09		
						18	35.09		
						19	34.09		
						20	33.09		
						21	32.09		
						22	31.09		
						23	30.09		
						24	29.09		
						25	28.09		
						26	27.09		
						27	26.09		
						28	25.09		
28.25 End of Borehole Practical DCPT refusal at 28.25m depth. (BH dry - July 24, 2018)									



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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FILE NO. PG3908

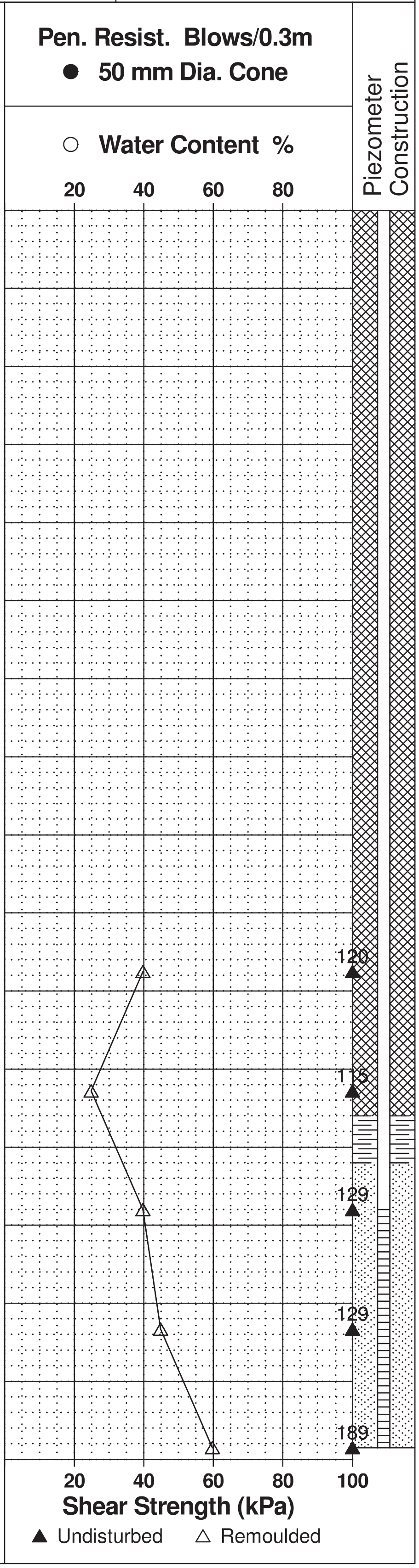
REMARKS

HOLE NO. BH 7

BORINGS BY CME 55 Power Auger

DATE July 18, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE		AU	1			0	52.67
						1	51.67
						2	50.67
Very stiff, brown **SILTY CLAY** - grey by 3.8m depth						3	49.67
						4	48.67
						5	47.67
						6	46.67
						7	45.67
						8	44.67
						9	43.67
						10	42.67
						11	41.67
						12	40.67
						13	39.67
						14	38.67
						15	37.67
15.85							
						16	36.67



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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

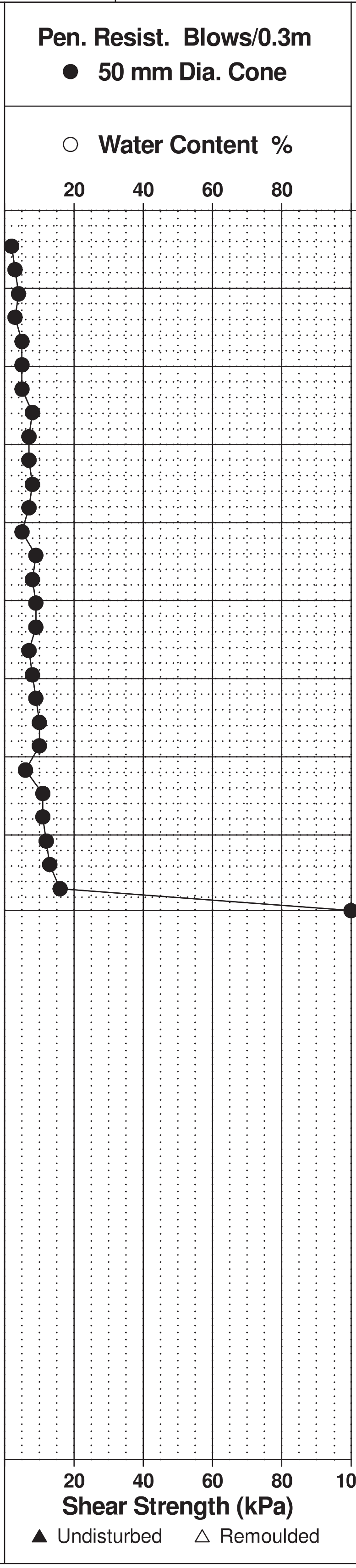
BORINGS BY CME 55 Power Auger

DATE July 18, 2018

FILE NO. PG3908

HOLE NO. BH 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE								
Dynamic Cone Penetration Test commenced at 15.85m depth.						16	36.67	
						17	35.67	
						18	34.67	
						19	33.67	
						20	32.67	
						21	31.67	
						22	30.67	
						23	29.67	
						24	28.67	
24.97								
End of Borehole Practical DCPT refusal at 24.97m 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

FILE NO. PG3908

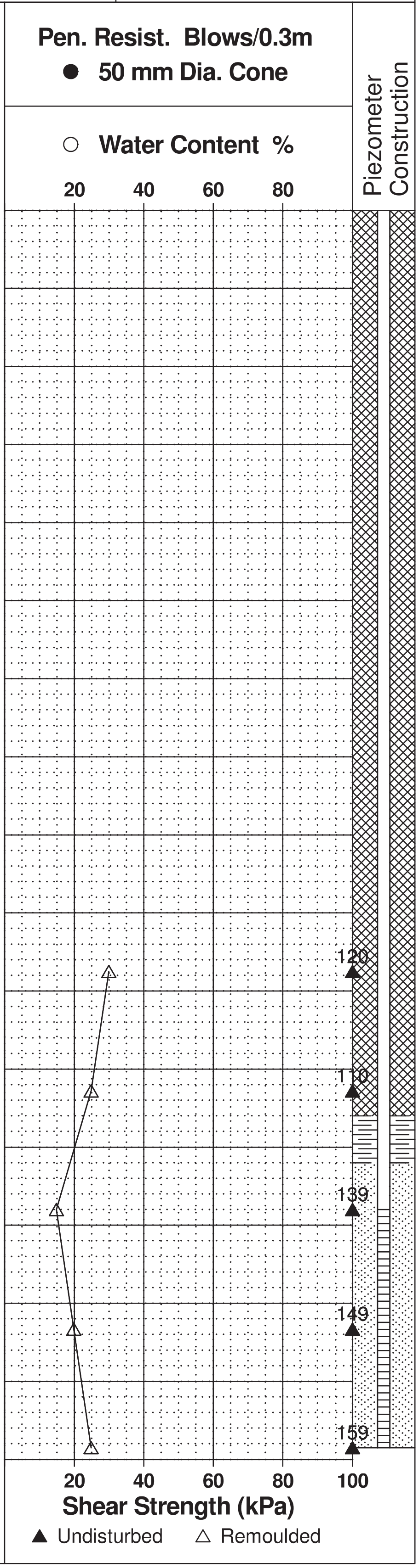
REMARKS

HOLE NO. BH 8

BORINGS BY CME 55 Power Auger

DATE July 18, 2018

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	52.25
	AU	1				
					1	51.25
	SS	2	75	9		
					2	50.25
					3	49.25
					4	48.25
					5	47.25
					6	46.25
Very stiff, brown **SILTY CLAY**						
					7	45.25
- grey by 7.5m depth						
					8	44.25
					9	43.25
					10	42.25
					11	41.25
					12	40.25
					13	39.25
					14	38.25
					15	37.25
15.85						
					16	36.25



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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 18, 2018

FILE NO. PG3908

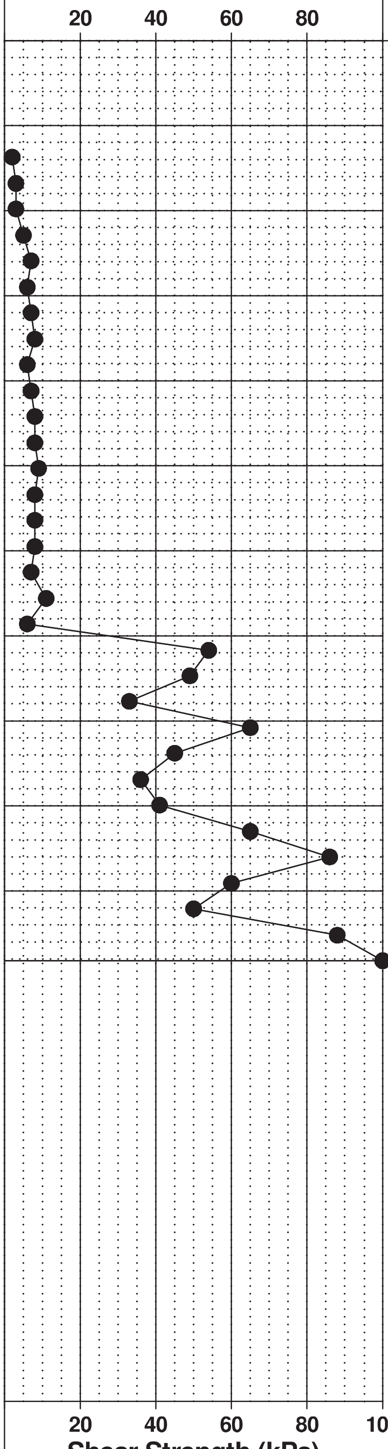
HOLE NO. BH 8

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.						16	36.25
						17	35.25
						18	34.25
						19	33.25
						20	32.25
						21	31.25
						22	30.25
						23	29.25
						24	28.25
						25	27.25
						26	26.25
26.82							
End of Borehole Practical DCPT refusal at 26.82m depth. (BH dry - July 24, 2018)							

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

○ Water Content %

Piezometer Construction



▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

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

DATUM Geodetic

FILE NO. PG3908

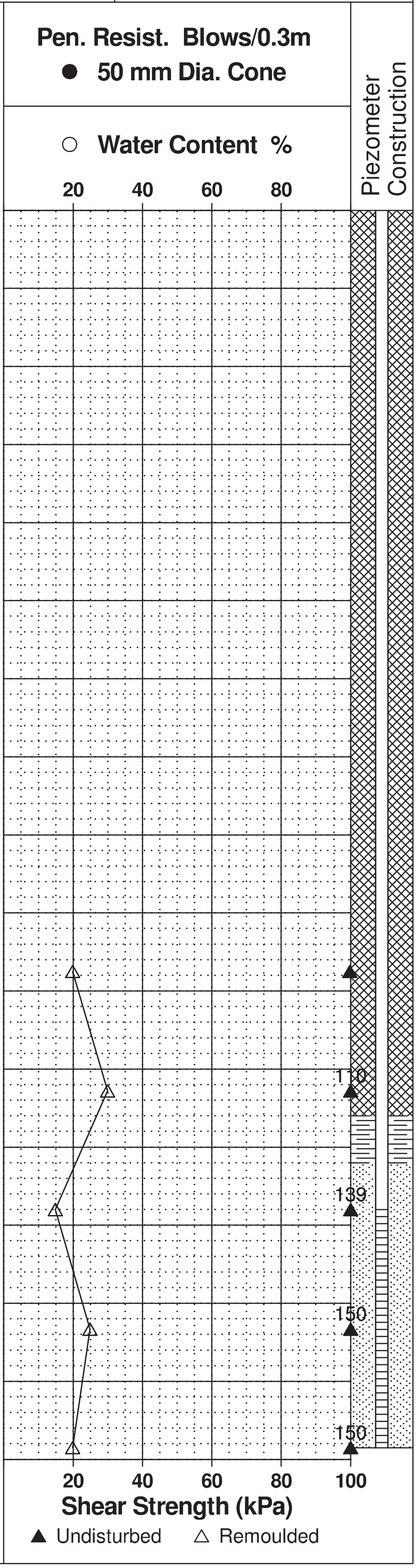
REMARKS

HOLE NO. BH 9

BORINGS BY CME 55 Power Auger

DATE July 19, 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
FILL: Brown silty sand with gravel, concrete, cobbles, boulders 0.51		AU	1			0	52.81
						1	51.81
						2	50.81
Very stiff, brown **SILTY CLAY**						3	49.81
- grey by 3.8m depth						4	48.81
						5	47.81
						6	46.81
						7	45.81
						8	44.81
						9	43.81
						10	42.81
						11	41.81
						12	40.81
						13	39.81
						14	38.81
						15	37.81
15.85						16	36.81



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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 19, 2018

FILE NO. PG3908

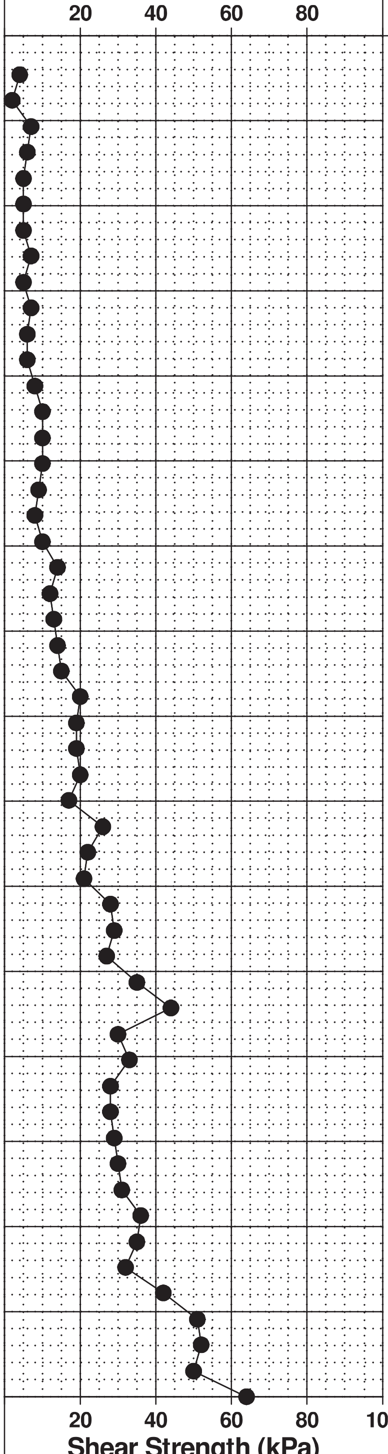
HOLE NO. BH 9

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.						16	36.81
						17	35.81
						18	34.81
						19	33.81
						20	32.81
						21	31.81
						22	30.81
						23	29.81
						24	28.81
						25	27.81
						26	26.81
						27	25.81
						28	24.81
						29	23.81
						30	22.81
						31	21.81
32.00						32	20.81

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

○ Water Content %

Piezometer Construction



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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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 19, 2018

FILE NO. PG3908

HOLE NO. BH 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			20 40 60 80	
End of Borehole (BH dry upon completion)									

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)

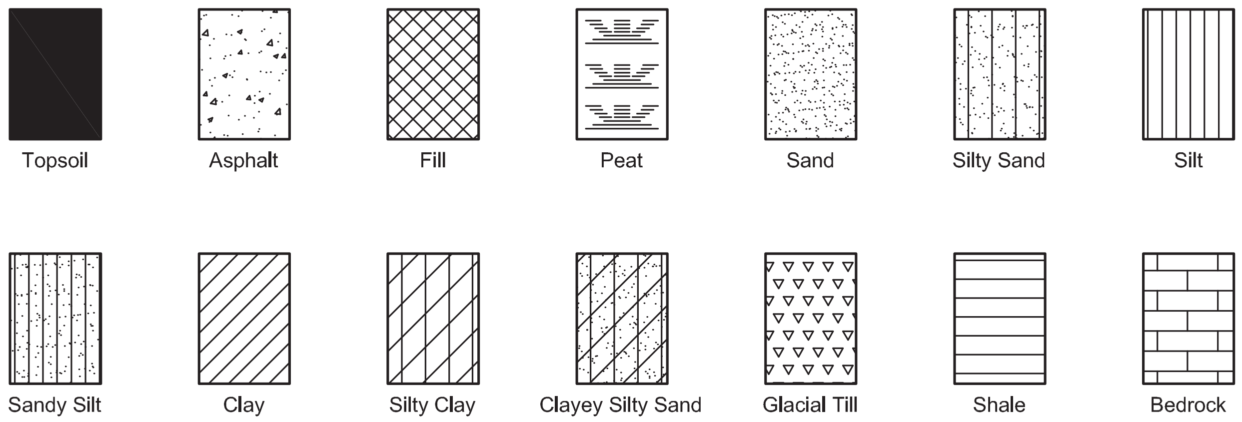
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

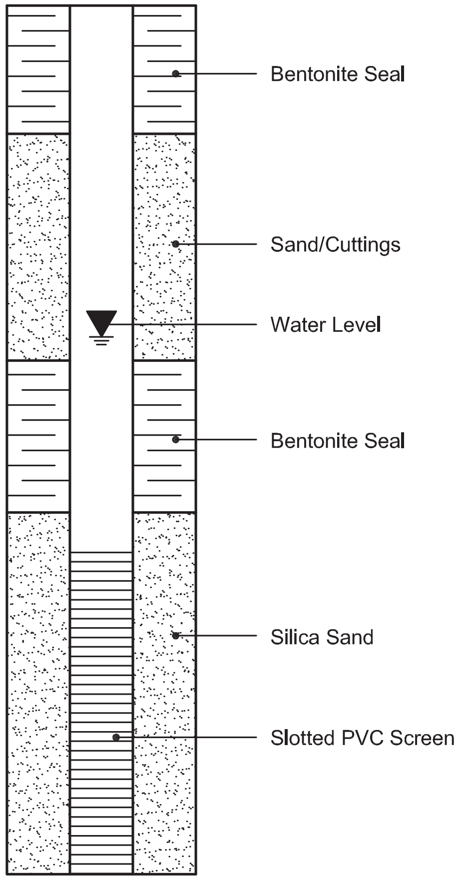
SYMBOLS AND TERMS (continued)

STRATA PLOT

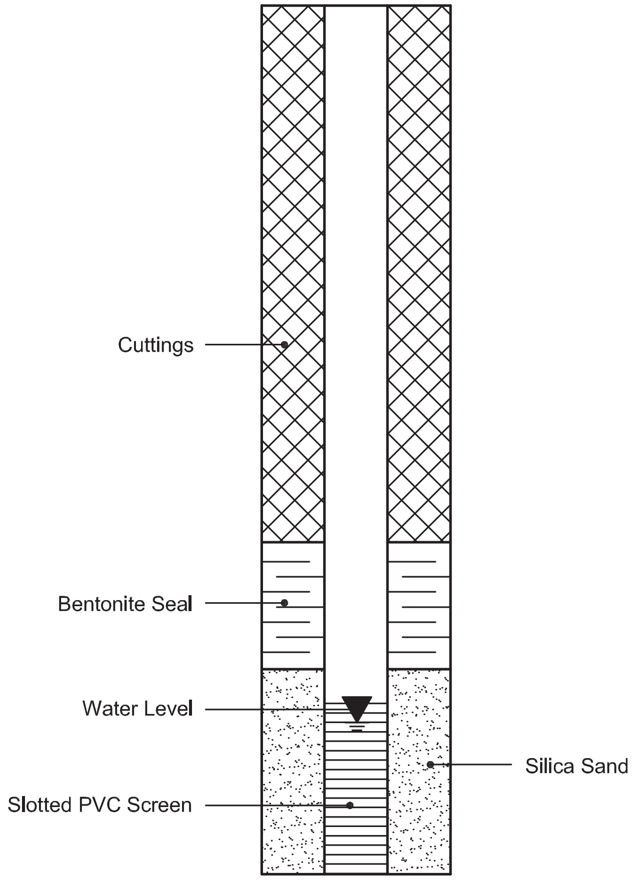


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis
Client: Paterson Group Consulting Engineers
Client PO: 24716

Report Date: 25-Jul-2018
Order Date: 20-Jul-2018
Project Description: PG3908

	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

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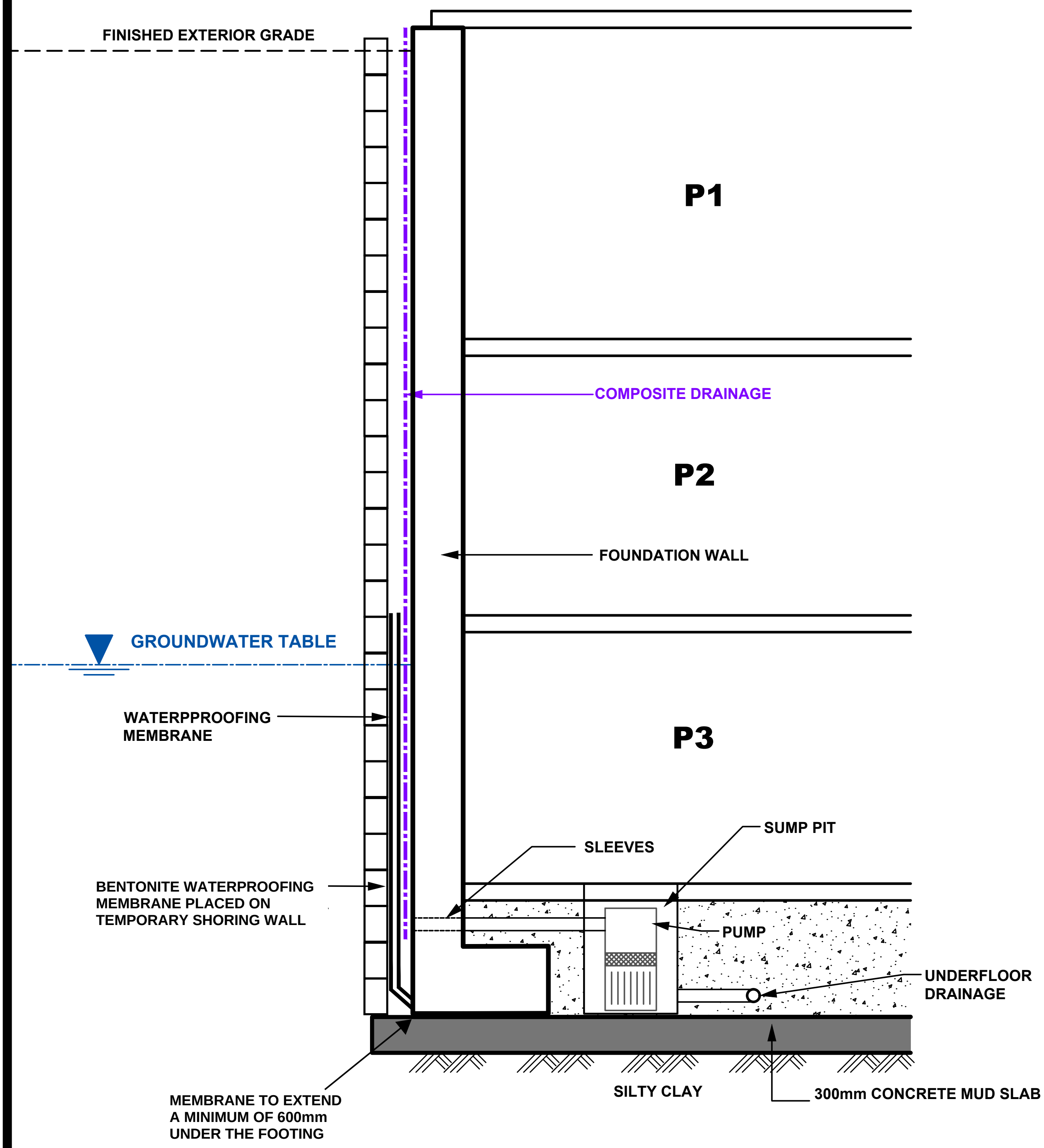

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www.patersongroup.ca

BRIGIL

PROPOSED MULTI-STOREY BUILDING
PETRIE LANDING
OTTAWA, ONTARIO

Title:

WATER SUPPRESSION SYSTEM

Date: **01/2021**

Report No.: **PG3908-2**

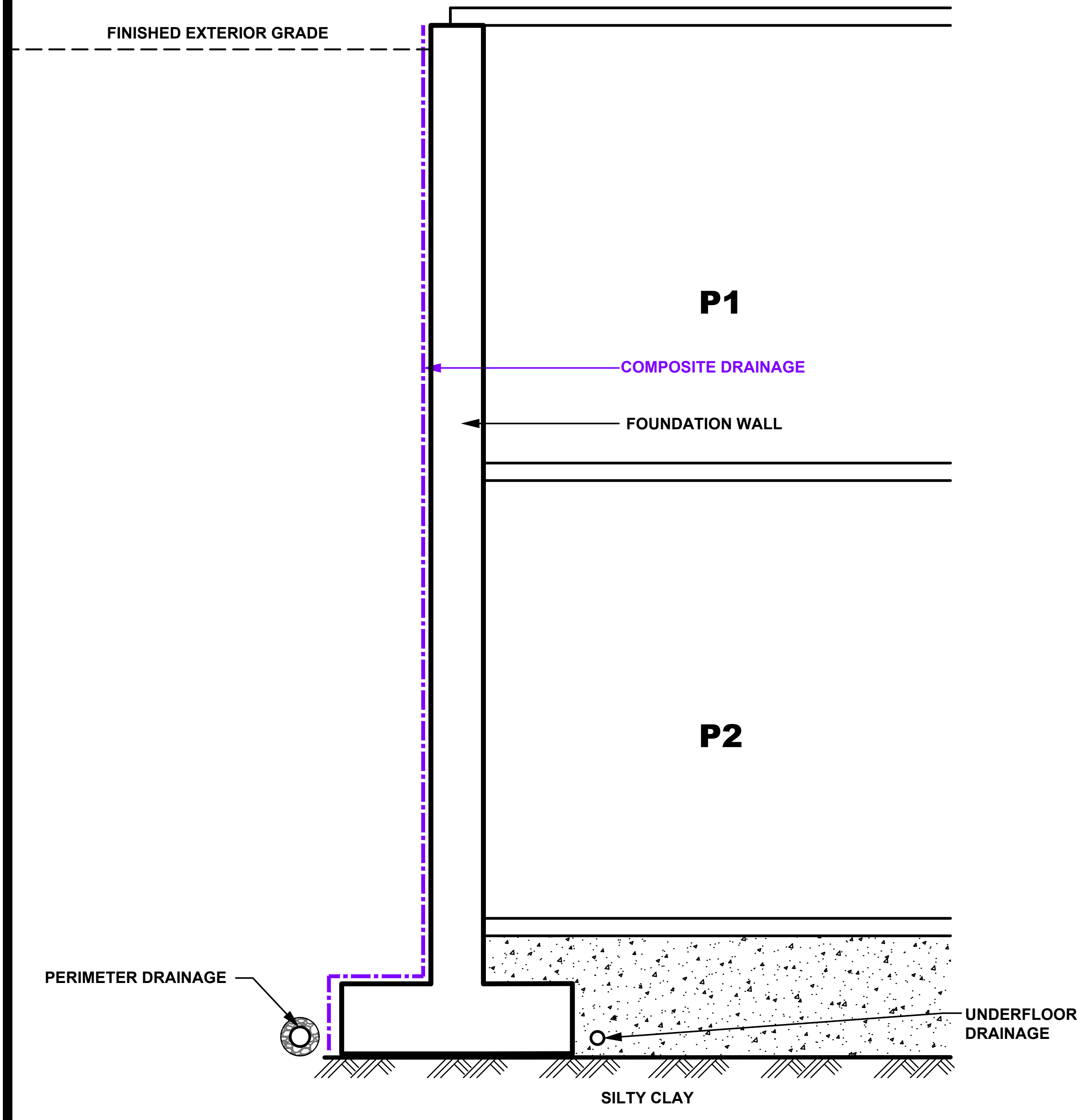
Scale: **N.T.S**

Drawing No.: **FIGURE 2**

Drawn by: **NFRV**

Checked by: **JV**

c:\users\nicholasv\documents\old files\5370\pg5370-figure 4 - water supresion system.dwg



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BRIGIL

PROPOSED MULTI-STOREY BUILDING
PETRIE LANDING
OTTAWA, ONTARIO

Title:

FOUNDATION DRAINAGE DETAIL

Date: **03/2021**

Report No.: **PG3908-2**

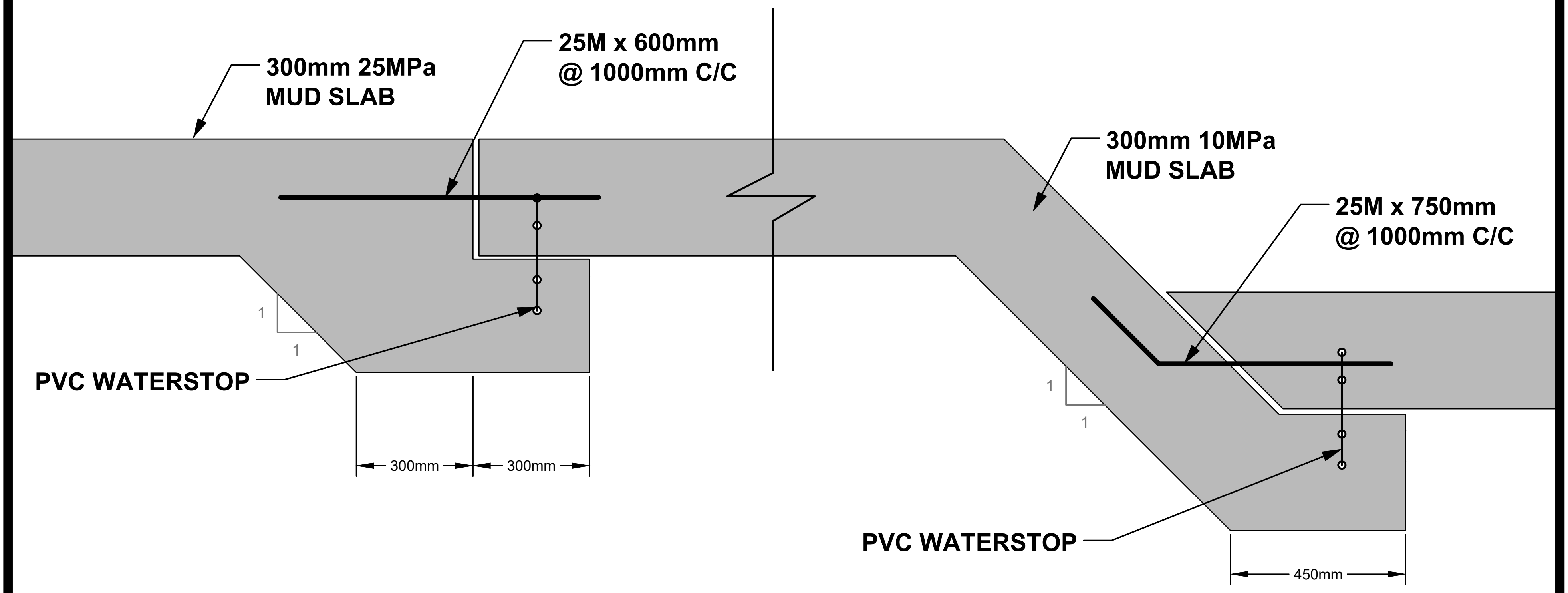
Scale: **N.T.S**

Drawing No.: **FIGURE 3**

Drawn by: **NFRV**

Checked by: **JV**

c:\users\nicholasv\documents\old files\pg3908\pg3908-foundation drainage detail.dwg



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BIRGIL CONSTRUCTION

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

OTTAWA, ONTARIO

Title:

MUD SLAB COLD JOINT TRANSITION DETAIL

Scale: N.T.S.	Date: 11/2021
Drawn by: NFRV	Report No.: PG3908-2
Checked by: JV	Drawing No.: FIGURE 4
Approved by: DJG	Revision No.:

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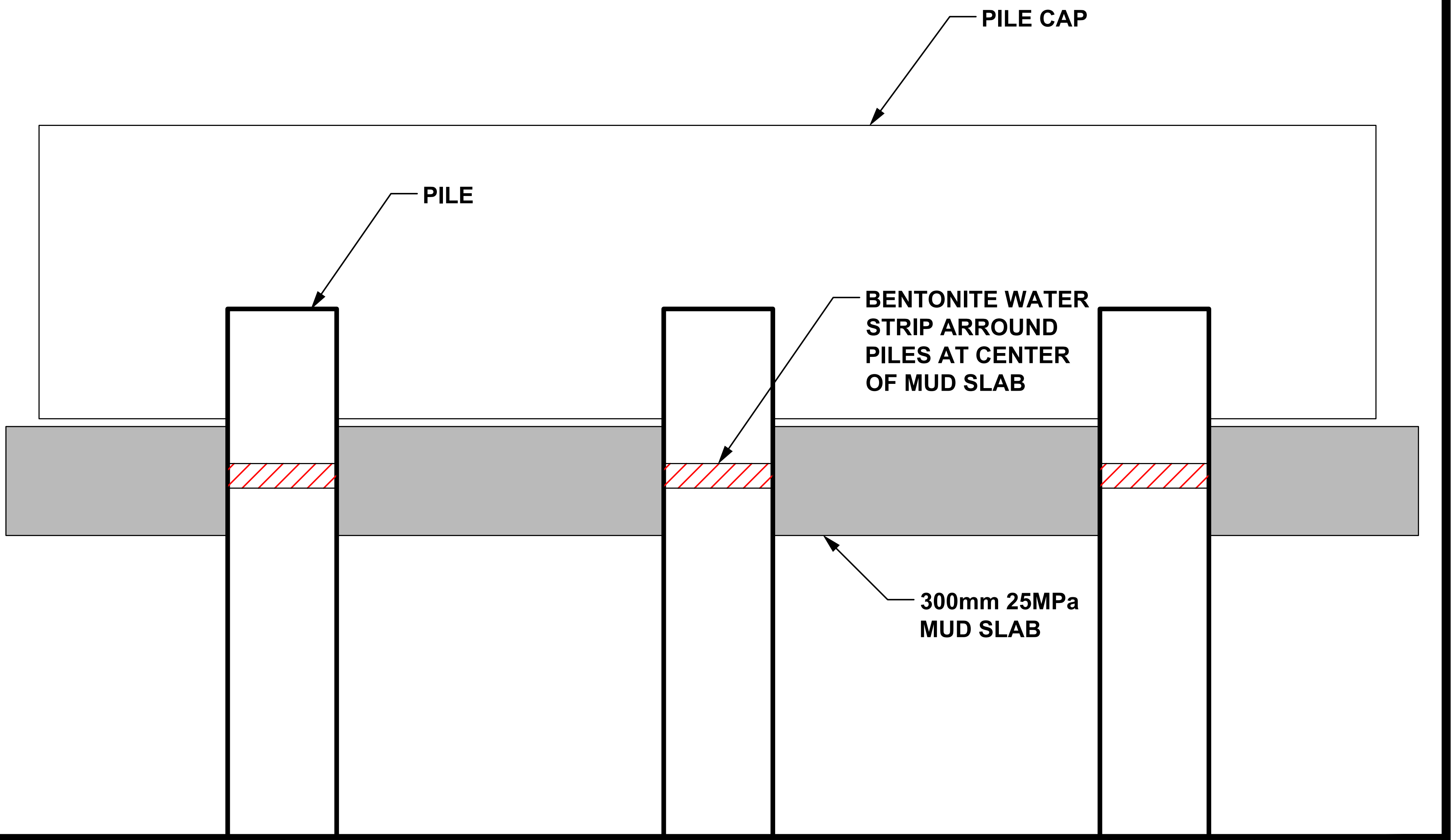

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PROPOSED MULTI-STOREY BUILDING
TOWER 4, 5A & 5B - PETRIE'S LANDING - INLET
OTTAWA, ONTARIO

Title:

PILE WATERPROOFING DETAIL

Scale: N.T.S.	Date: 11/2021
Drawn by: NFRV	Report No.: PG3908-2
Checked by: JV	Drawing No.: FIGURE 5
Approved by: DJG	Revision No.:

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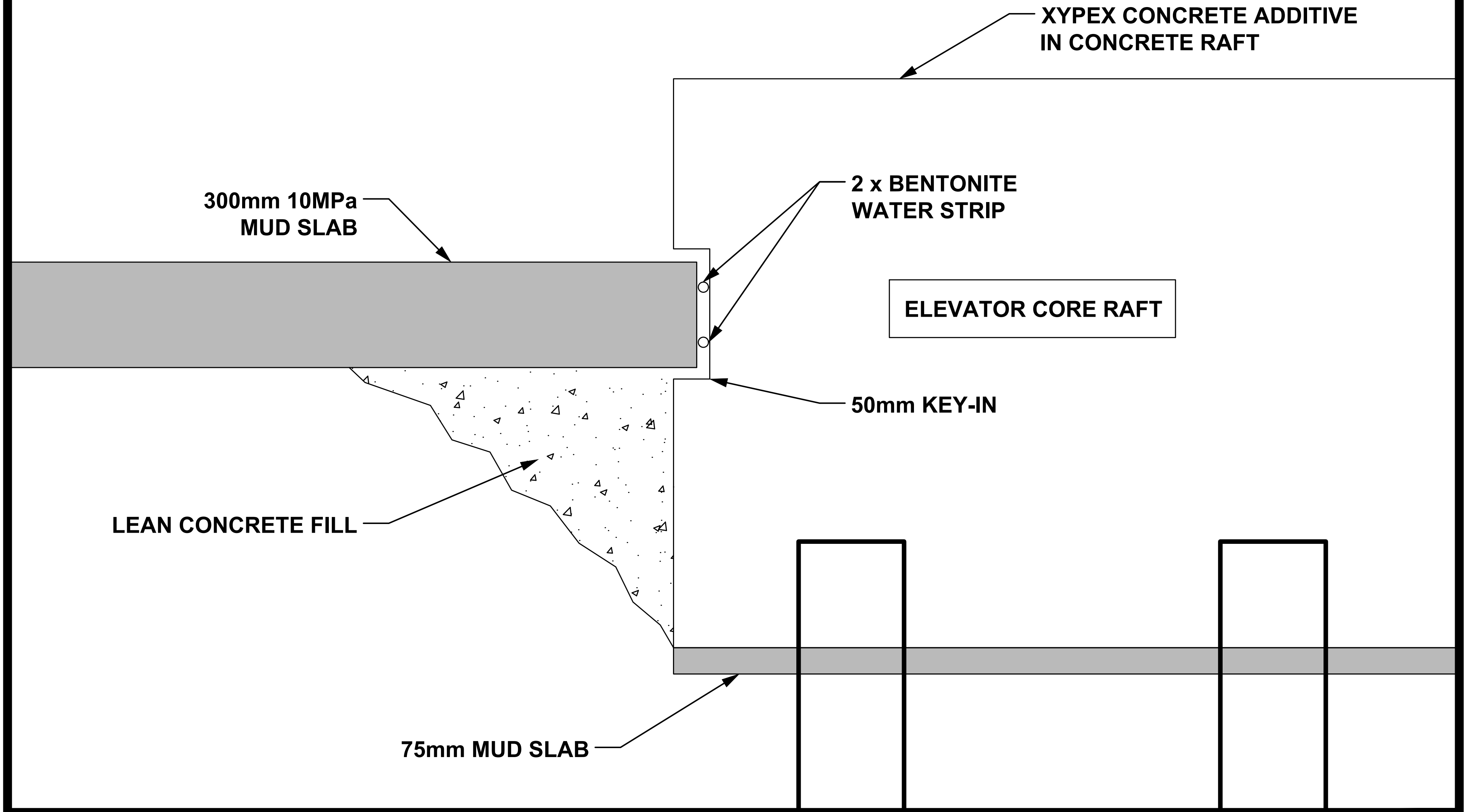

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BIRGIL CONSTRUCTION

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

OTTAWA, ONTARIO

Title:

ELEVATOR SHAFT DETAIL

Scale: N.T.S.	Date: 11/2021
Drawn by: NFRV	Report No.: PG3908-2
Checked by: JV	Drawing No.: FIGURE 6
Approved by: DJG	Revision No.:

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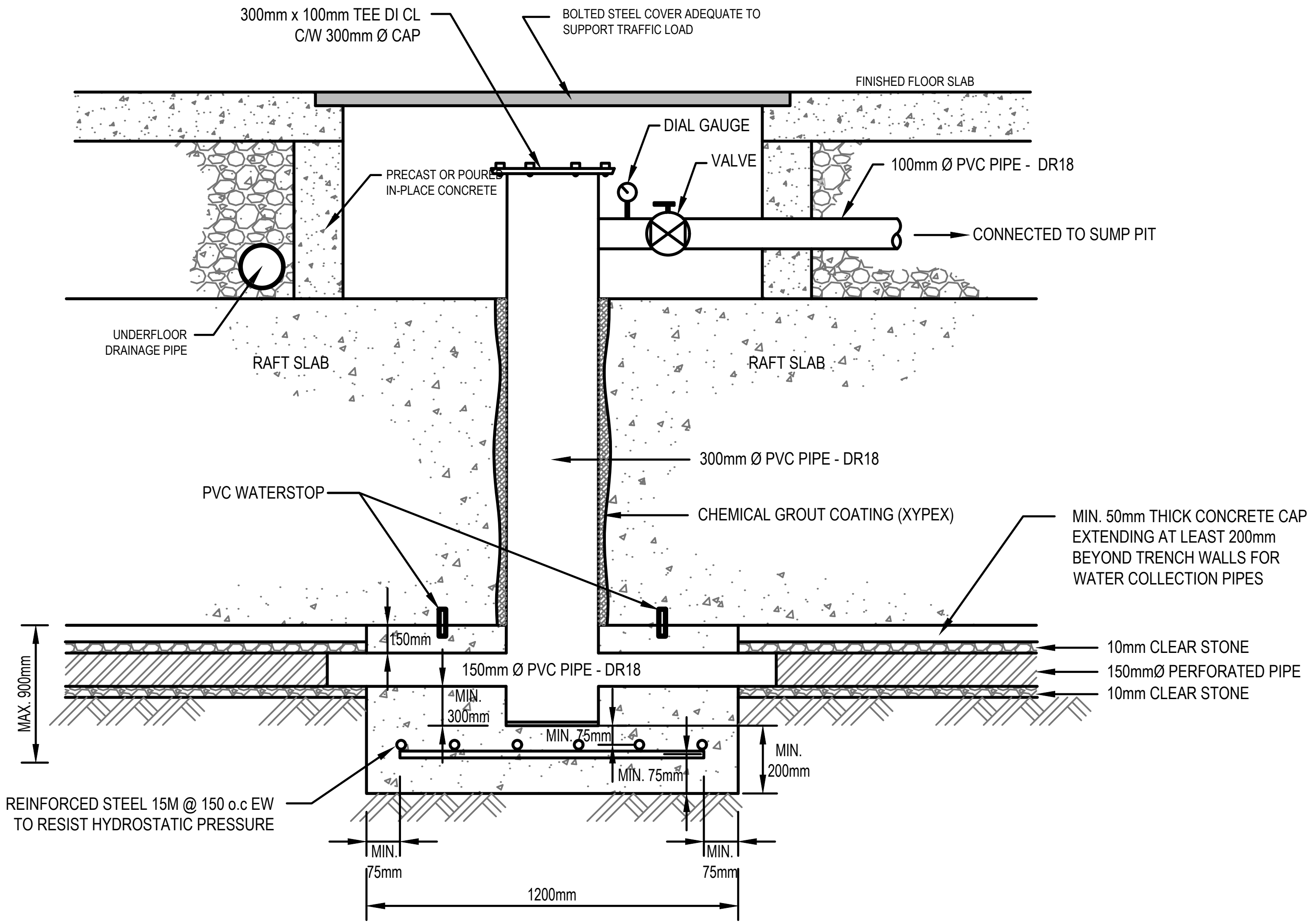

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BIRGIL CONSTRUCTION

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

OTTAWA, ONTARIO

Title:

PRESSURE RELIEF CHAMBER

Scale: N.T.S.
Date: 11/2021
Drawn by: NFRV
Report No.: PG3908-2
Checked by: JV
Drawing No.: FIGURE 7
Approved by: DJG
Revision No.:

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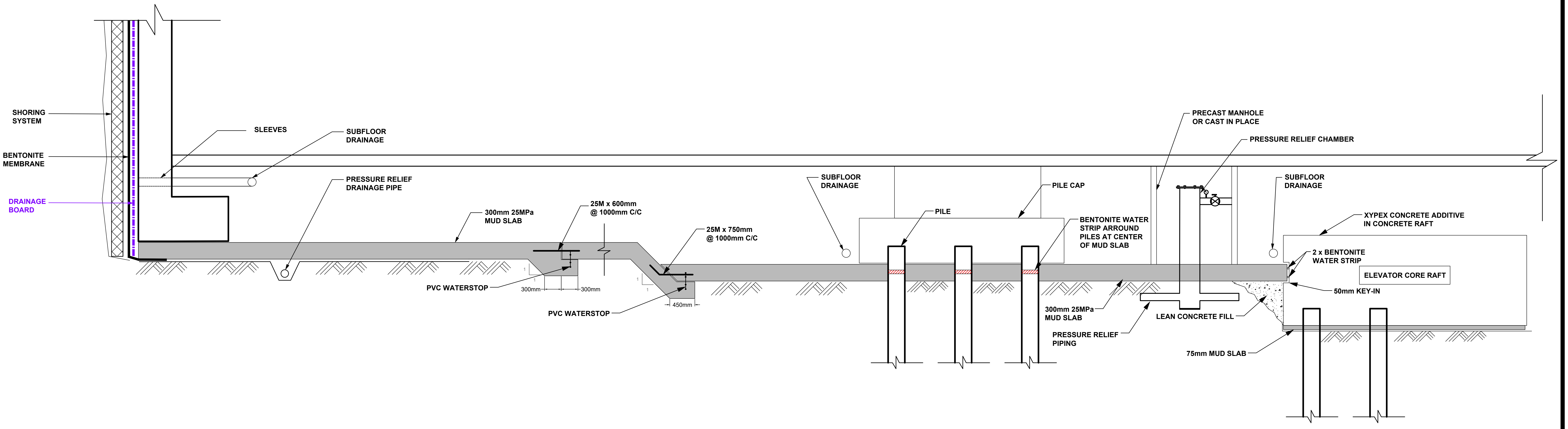

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Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

BIRGIL CONSTRUCTION

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

Title:

WATERPROOFING SYSTEM

Stamp:

Scale: N.T.S.
Drawn by: NFRV
Checked by: JV
Approved by: DJG
Date: 11/2021

Report No.: PG3908-2
Drawing No.: FIGURE 8
Revision No.:

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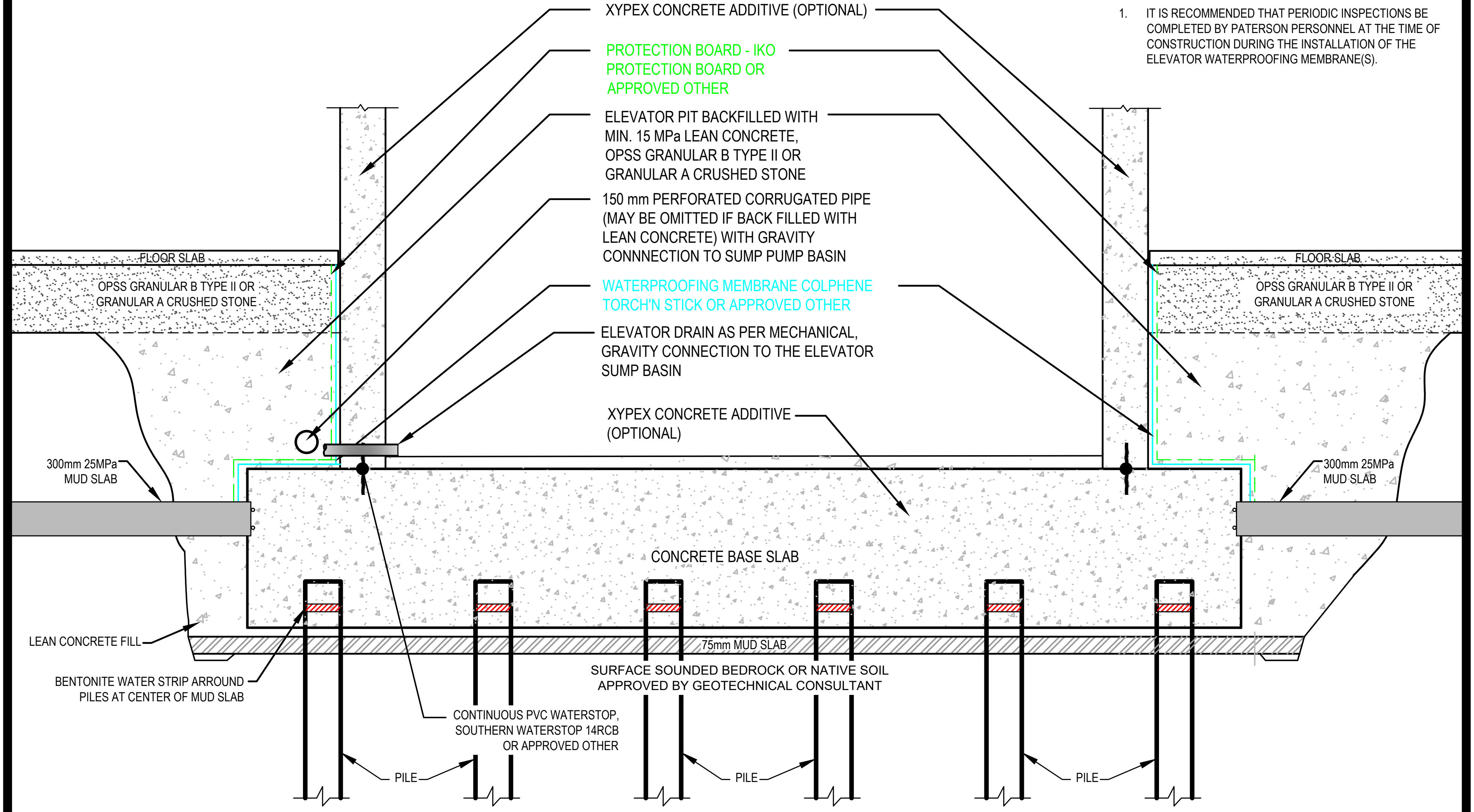
NOTES:

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).



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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:** 11/2021
Drawn by: NFRV	**Report No.:** PG3908-2
Checked by: JV	**Drawing No.:** FIGURE 9
Approved by: DJG	**Revision No.:**

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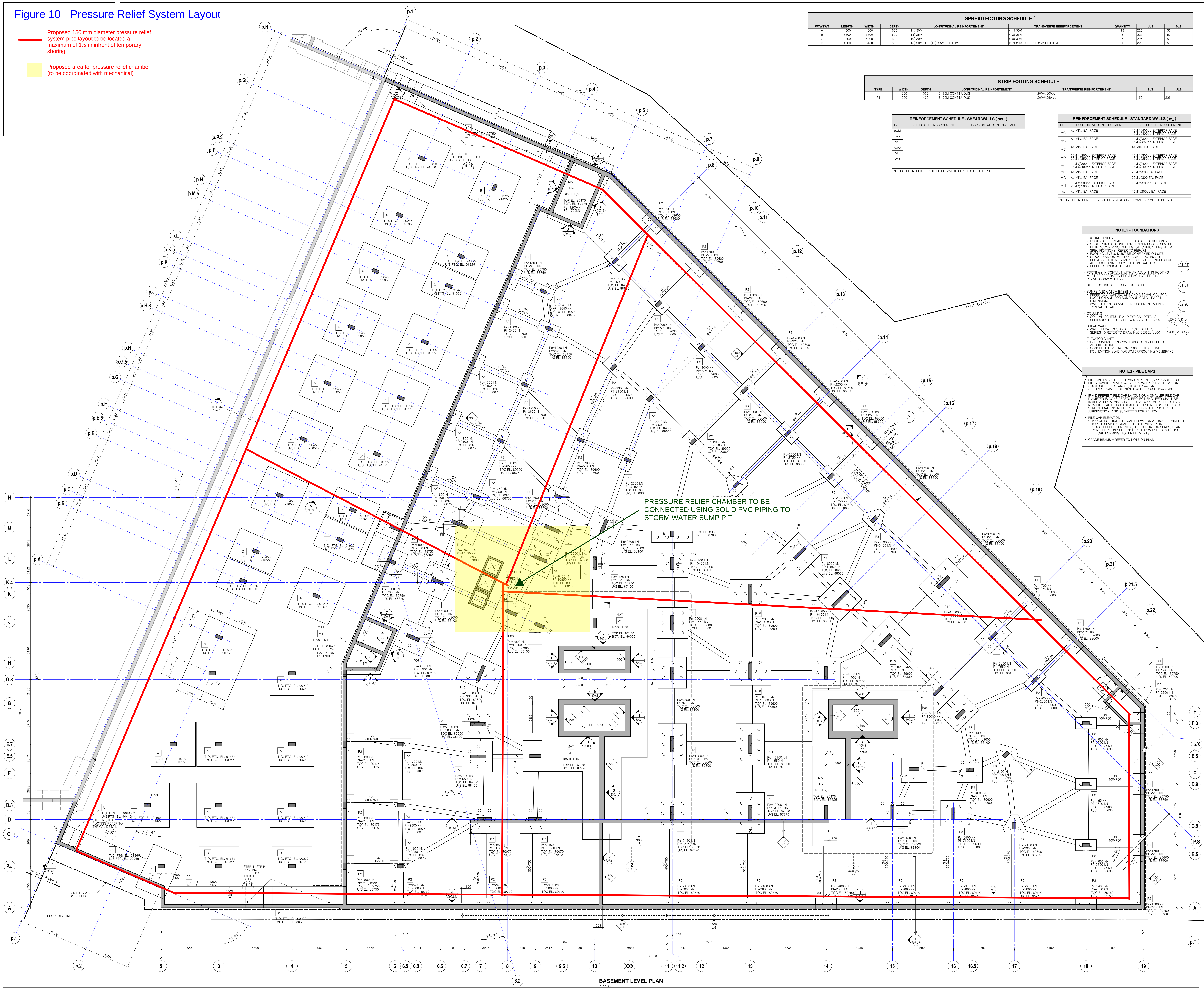
Figure 10 - Pressure Relief System Layout

🟥 Proposed 150 mm diameter pressure relief system pipe layout to be located a maximum of 1.5 m infront of temporary shoring

🟨 Proposed area for pressure relief chamber (to be coordinated with mechanical)



PRESSURE RELIEF CHAMBER TO BE CONNECTED USING SOLID PVC PIPING TO STORM WATER SUMP PIT

SPREAD FOOTING SCHEDULE								
WTWTWT	LENGTH	WIDTH	DEPTH	LONGITUDINAL REINFORCEMENT	TRANSVERSE REINFORCEMENT	QUANTITY	ULS	SLS
A	4000	4000	600	(11) 30M	(11) 30M	18	225	150
B	3600	3600	500	(13) 25M	(13) 25M	3	225	150
C	2800	4200	600	(10) 30M	(10) 30M	7	225	150
D	4500	6450	800	(15) 20M TOP (13)–25M BOTTOM	(17) 20M TOP (21)–25M BOTTOM	1	225	150

STRIP FOOTING SCHEDULE						
TYPE	WIDTH	DEPTH	LONGITUDINAL REINFORCEMENT	TRANSVERSE REINFORCEMENT	SLS	ULS
	1800	300	(6) 20M CONTINUOUS	20M@300oc		
S1	1900	400	(8) 20M CONTINUOUS	20M@250 oc	150	225

REINFORCEMENT SCHEDULE - SHEAR WALLS (sw_)		
TYPE	VERTICAL REINFORCEMENT	HORIZONTAL REINFORCEMENT
swM		
swN		
swP		
swQ		
swR		
swS		

NOTE: THE INTERIOR FACE OF ELEVATOR SHAFT IS ON THE PIT SIDE

REINFORCEMENT SCHEDULE - STANDARD WALLS (w_)		
TYPE	HORIZONTAL REINFORCEMENT	VERTICAL REINFORCEMENT
wA	As MIN. EA. FACE	15M @400oc EXTERIOR FACE 15M @400oc INTERIOR FACE
wB	As MIN. EA. FACE	15M @300oc EXTERIOR FACE 15M @250oc INTERIOR FACE
wC	As MIN. EA. FACE	As MIN. EA. FACE
wD	20M @250oc EXTERIOR FACE 20M @350oc INTERIOR FACE	15M @300oc EXTERIOR FACE 15M @250oc INTERIOR FACE
wE	15M @300oc EXTERIOR FACE 15M @400oc INTERIOR FACE	15M @400oc EXTERIOR FACE 15M @400oc INTERIOR FACE
wF	As MIN. EA. FACE	25M @200 EA. FACE
wG	As MIN. EA. FACE	20M @300 EA. FACE
wH	15M @300oc EXTERIOR FACE 20M @200oc INTERIOR FACE	15M @200oc EA. FACE
wJ	As MIN. EA. FACE	15M@250oc EA. FACE

NOTE: THE INTERIOR FACE OF ELEVATOR SHAFT WALL IS ON THE PIT SIDE

NOTES - FOUNDATIONS

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  - UPWARD ADJUSTMENT OF SOME FOOTINGS IS PERMISSIBLE IF MECHANICAL SERVICES UNDER SLAB ARE COORDINATED BY THE CONTRACTOR
  - REFER TO TYPICAL DETAIL 01.04
- FOOTINGS IN CONTACT WITH AN ADJOINING FOOTING MUST BE SEPARATED FROM EACH OTHER BY A PLYWOOD 25mm THICK
- STEP FOOTING AS PER TYPICAL DETAIL 01.01
- SUMPS AND CATCH BASSINS
  - REFER TO ARCHITECTURE AND MECHANICAL FOR LOCATION AND FOR SUMP AND CATCH BASSIN DIMENSIONS
  - WALL THICKNESS AND REINFORCEMENT AS PER TYPICAL DETAIL 02.20
- COLUMNS
  - COLUMN SCHEDULE AND TYPICAL DETAILS SERIES 09 REFER TO DRAWINGS SERIES S200 200.0 201.x
- SHEAR WALLS
  - WALL ELEVATIONS AND TYPICAL DETAILS SERIES 10 REFER TO DRAWINGS SERIES S300 300.0 30x.x
- ELEVATOR SHAFT
  - FOR DRAINAGE AND WATERPROOFING REFER TO ARCHITECTURE
  - CONCRETE LEVELING PAD 100mm THICK UNDER FOUNDATION SLAB FOR WATERPROOFING MEMBRANE

NOTES - PILE CAPS

- PILE CAP LAYOUT AS SHOWN ON PLAN IS APPLICABLE FOR PILES HAVING AN ALLOWABLE CAPACITY (SLS) OF 1200 kN, (FACTORED RESISTANCE (ULS) OF 1440 kN),
  - PILES OF 245mm OUTSIDE DIAMETER AND 13mm WALL
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- PILE CAP ELEVATION
  - TOP OF INTERIOR PILE CAP ELEVATION AT 450mm UNDER THE TOP OF SLAB ON GRADE AT ITS LOWEST POINT
  - NEAR DEEPER ELEMENTS (EX. FOUNDATION SLABS) PLAN CONSTRUCTION SEQUENCE TO ALLOW FOR BACKFILLING BEFORE FORMING HIGHER ELEMENTS
- GRADE BEAMS – REFER TO NOTE ON PLAN

No.	Date	Description
1	2021.11.01	COORDINATION 70%
	2021.09.10	COORDINATION 60%

ISSUES

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Civil
EXP SERVICES INC
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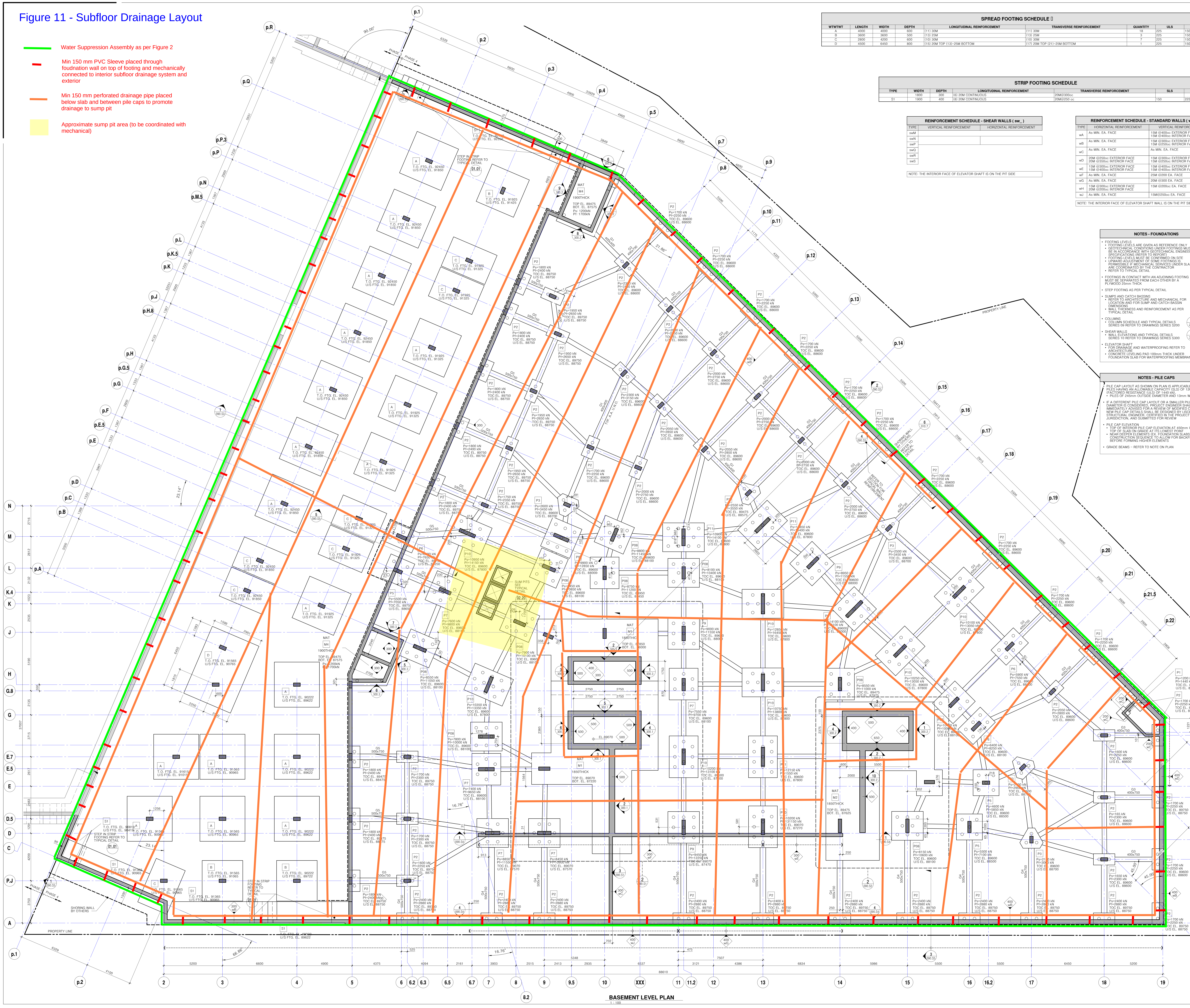
Project
PETRIE'S LANDING I PHASE 4

Drawing Title
FOUNDATIONS

Scale AS NOTED
Drafting -
Design -
Check -
Project No. 19-049
Plot Date 2021-11-01 16:19:14
Drawing No. S090.0
Issue No. 1

Figure 11 - Subfloor Drainage Layout

🟩	Water Suppression Assembly as per Figure 2
🟥	Min 150 mm PVC Sleeve placed through foundation wall on top of footing and mechanically connected to interior subfloor drainage system and exterior
🟧	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)



SPREAD FOOTING SCHEDULE								
WTWTWT	LENGTH	WIDTH	DEPTH	LONGITUDINAL REINFORCEMENT	TRANSVERSE REINFORCEMENT	QUANTITY	ULS	SLS
A	4000	4000	600	(11) 30M	(11) 30M	18	225	150
B	3600	3600	500	(13) 25M	(13) 25M	3	225	150
C	2800	4200	600	(10) 30M	(10) 30M	7	225	150
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swN		
swP		
swQ		
swR		
swS		

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wD	20M @250oc EXTERIOR FACE 20M @350oc INTERIOR FACE	15M @300oc EXTERIOR FACE 15M @250oc INTERIOR FACE
wE	15M @300oc EXTERIOR FACE 15M @400oc INTERIOR FACE	15M @400oc EXTERIOR FACE 15M @400oc INTERIOR FACE
wF	As MIN. EA. FACE	25M @200 EA. FACE
wG	As MIN. EA. FACE	20M @300 EA. FACE
wH	15M @300oc EXTERIOR FACE 20M @200oc INTERIOR FACE	15M @200oc EA. FACE
wJ	As MIN. EA. FACE	15M@250oc EA. FACE

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630, bol. René – Lévesque O. 32e étages, Montréal QC
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Seals

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Project
PETRIE'S LANDING I
PHASE 4

Drawing Title
FOUNDATIONS

Scale	AS NOTED	Project No.	19-049
Drafting	-		
Design	-	Plot Date	2021-11-01 16:19:14
Check	-		

Drawing No. S090.0
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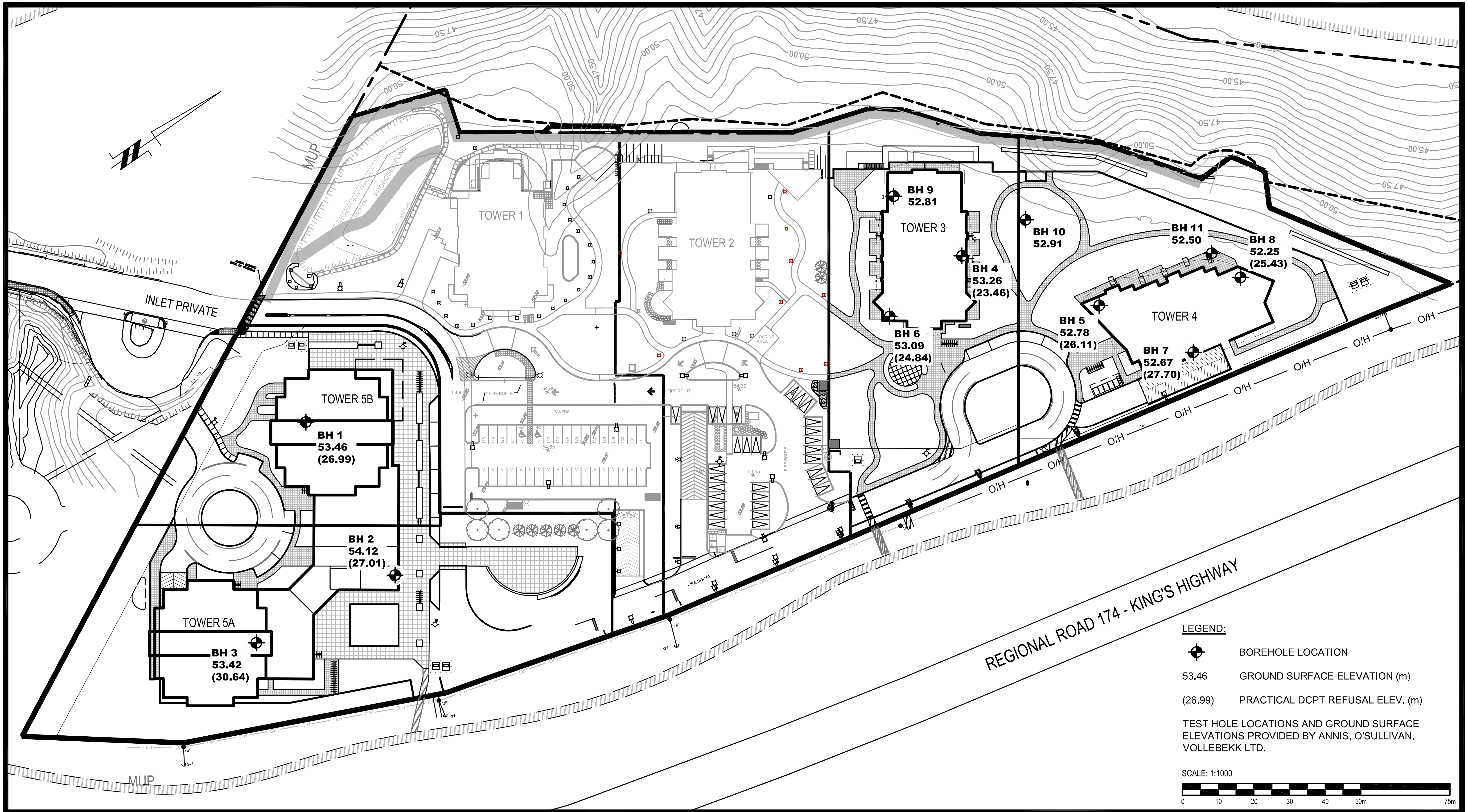

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NO.	REVISIONS	DATE	INITIAL
3	BH 10 AND BH 11 ADDED	20/09/2019	DJG
2	BASE PLAN UPDATED	17/07/2019	FA
1	BASE PLAN UPDATED	25/04/2019	FA

BRIGIL CONSTRUCTION
GEOTECHNICAL INVESTIGATION
PROP. MULTI-BUILDING DEVELOPMENT - PETRIE LANDING
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:1000
Date: 08/2018
Drawn by: MPG
Report No.: PG3908-1
Checked by: FA
Dwg. No.: **PG3908-1**
Approved by: DJG
Revision No.: 3

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APPENDIX 3

TYPICAL FOUNDATION SLEEVE INSTALLATION

Typical 150 mm Diameter Sleeve Installation

Photo 1 – Step 1: It is recommended that the upper 1/3 of the 150 mm drainage sleeve be cut at a 45 degree angle to hydraulically connect the composite foundation drainage board to the interior and underfloor drainage system.

Photo 2 – Step 2: It is recommended that the 150 mm diameter drainage sleeve be installed by carefully cutting an 'X' shaped incision through the composite foundation drainage and inserting the 150 mm diameter drainage sleeve inside the 'X' by pulling the four (4) triangular flaps towards the installer.

Photo 3 – Step 3: Apply a suitable primer prior to the placement of the adhesive tape such as 3M tape, WP200 BlueSkine or equivalent.

Photo 4 – Step 4: An adhesive such as 3M tape, BlueSkin, or equivalent be utilized to seal the 150 mm drainage sleeve to the composite foundation drainage board to act as a barrier in preventing concrete from blocking connection during the placement of the exterior concrete foundation wall.

Photo 5 – Step 5: As an additional precaution, it is also recommended that an adhesive tape be placed on the interior outlet end of the drainage sleeve between the temporary form work to further prevent concrete from entering the drainage sleeve during the placement of concrete. Once the temporary form work has been removed, the adhesive tape can be cut away to allow groundwater to have a positive gravity connection to the interior perimeter and underfloor drainage system.