PATERSON GROUP

Geotechnical Investigation

Proposed High-Rise Buildings

267 O'Connor Street
Ottawa, Ontario

Prepared for Taggart Realty Management

Report PG4985-1 Revision 4 dated February 3, 2025

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Taggart Realty Management to conduct a geotechnical investigation for the proposed high-rise buildings to be located at 267 O'Connor Street in the City of Ottawa (reference should be made to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

- Determine the subsoil and groundwater conditions at this site by means of boreholes.
- 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 carried out in a separate investigation and was not part of the scope of work for this geotechnical investigation.

2.0 Proposed Development

Based on the current conceptual plans, it is understood that a 27-storey (Phase 1) and 25-storey (Phase 2) residential buildings are proposed to be constructed at the subject site. Phase 1 is understood to occupy the northern half of the site while Phase 2 will occupy the southern half of the site. It is also understood four levels of shared underground parking are proposed to occupy the majority of the footprint of the subject site.

At-grade parking areas, landscaped areas, and walkways along with access lanes are anticipated as part of the proposed development. The proposed buildings are also anticipated to be municipally serviced. The existing building throughout the northwestern portion of the subject site is anticipated to be demolished as part of the proposed development. It is further understood that Phase 1 of the development will be constructed and completed prior to the commencement of Phase 2.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation completed by this firm was carried out on November 24, 2012. At that time, three (3) boreholes were advanced to a maximum depth of 14.2 m below existing grade. A supplemental field investigation was carried out on March 22, 2014. At that time, 3 boreholes were advanced to a maximum depth of 14.3 m below existing grade. A Phase II Environmental Site Assessment (ESA) was conducted in 2020 and consisted of advancing one borehole to a maximum depth of 3.4 m below existing grade at the aforementioned site.

The borehole locations were distributed in a manner to provide general coverage of the subject site taken into consideration existing structures, utilities and other site features. The locations of the boreholes are shown on Drawing PG4985-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a truck-mounted auger drill and portable rigs operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of auguring to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

The soil samples were recovered from the auger flights and using a 50 mm diameter split-spoon sampler. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the auger and split-spoon samples and auger were recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

Overburden thickness was evaluated during the course of the investigation by dynamic cone penetration test (DCPT) at boreholes BH 1 through BH 3 and BH 2-14 locations. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip. The steel drill rod is struck by 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.

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

Groundwater

Flexible standpipes were installed in boreholes BH 1 through BH 3 to monitor the groundwater levels subsequent to the completion of the sampling program. Groundwater monitoring wells were installed in boreholes BH 1-14, BH 3-14, and BH 4-20 to monitor the groundwater levels subsequent of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- 1.5 m of slotted 51 mm diameter PVC screen at the base of the boreholes.
- 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- No. 3 silica sand backfill within annular space around screen.
- 300 mm thick bentonite hole plug directly above PVC slotted screen.
- Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details. Groundwater level observations are discussed in Section 4.3 and are presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The borehole locations were determined by Paterson personnel taking into consideration the presence of underground and above-ground services. The location and ground surface elevation at each borehole location were surveyed by Paterson personnel.

The boreholes were surveyed with respect to a temporary benchmark (TBM), consisting of the top spindle of the fire hydrant located at the southeast corner of the intersection of O'Connor Street and Gilmour Street. A geodetic elevation of 71.88 m was assigned to this TBM. Borehole locations and ground surface elevations at the borehole locations are presented on Drawing PG4985-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

All soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by an existing six (6) storey building along with associated at grade asphalt parking areas. The ground surface across the subject site is relatively flat and approximately at grade with the surrounding roadways and adjacent properties.

The subject site is bordered to the north by Maclaren Street, to the west by O'Connor Street, to the south by Gilmour Street and to the east by a three-storey brick finished building with a stone block foundation. It should be noted that an existing building is located along the northwest property boundary of the subject site.

4.2 Subsurface Profile

Overburden

Generally, the soil profile encountered at the borehole locations consists of a pavement structure overlying a deep silty clay deposit. Practical refusal to DCPT was encountered at 18.6 m, 20.8 m, 18.3 m and 19.6 m depth below existing grade at BH 2-14, BH 1, BH 2 and BH 3, respectively. Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Based on available geological mapping, shale bedrock of the Billings Formation is present in this area with an overburden thickness ranging between 15 to 25 m.

4.3 Groundwater

Groundwater levels were recorded at boreholes location instrumented with a monitoring device. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1. The measured water levels within the monitoring wells are presented in Table 1 below:

Table 1 – Summary of Monitoring Well Water Levels				
Test Hole	**Ground Surface Elevation (m)**	**Measured Groundwater Level**		**Date Recorded**
		Depth (m)	**Elevation (m)**	
BH 1-14	71.95	8.03	63.92	April 2, 2014
BH 3-14	71.53	5.08	66.45	September 27, 2023
BH 4-20	68.60	0.19	68.41	September 27, 2023
Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS and referenced to a geodetic datum.				

The long-term groundwater table can also be estimated based on the consistency, colouring and moisture levels of the recovered soil samples at each borehole location. Therefore, the long-term groundwater table is estimated at a depth of 4 to 5 m below the existing grade. Groundwater levels are subject to seasonal fluctuations and therefore, groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed development. It is expected that the buildings will be founded over a raft foundation or deep foundations consisting of end-bearing pipe piles or caissons extending to the bedrock surface, or rock socketed caissons extending into the bedrock.

Should deep foundations be considered, the associated substructure could be utilized to provide foundation uplift resistance. Should the foundation uplift resistance capacities provided in Subsection 5.3 be insufficient for the foundation uplift loads, rock anchors may also be utilized to supplement the associated load resistance. The rock anchor design recommendations are discussed further in Subsection 5.7.

To complete the underground parking structure, a temporary shoring system will be required to support the overburden surrounding the subject site. The design of the shoring system should take into account the adjacent roads and buildings. Based on the depth of the lowest proposed parking level and the available space within Phase 2 of the proposed development, consideration can be given to open-cut excavation along the boundary between the proposed Phase 1 and Phase 2 during construction of Phase 1.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, asphalt and any deleterious fill should be removed from within the perimeter of the proposed building and other settlement sensitive structures. Foundation walls, underground services, and other construction debris should be entirely removed from within the perimeter of the buildings. Under paved areas, existing construction remnants, such as foundation walls, pipe ducts, etc., should be excavated to a minimum depth of 1 m below final grade.

Protection of Subgrade (Raft Foundation)

Where a raft foundation is utilized, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed, silty clay subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance to the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the silty clay to potential disturbance due to drying, and immediately (i.e., within 48 hours) of exposing the clay bearing medium. It should be understood that the mud slab alone is not considered sufficient to mitigate the potential for the migration of frost within the clay bearing medium if construction is undertaken during winter conditions.

Compacted Granular Fill Working Platform (Deep Foundation)

Should the proposed buildings be supported on a deep foundation, the use of heavy equipment would be required to install the piles and/or caissons. It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance.

A typical working platform could consist of 600 mm of Ontario Provincial Standard Specifications (OPSS) Granular B Type II crushed stone which is placed and compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in lifts not exceeding 300 mm in thickness.

Once the piles have been installed and cut off, the working platform can be regraded, and soil tracked in, or soil pumping up from the pile installation locations, can be bladed off and the surface can be topped up, if necessary, and recompacted to act as the substrate for further fill placement for the basement slab.

Fill Placement

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

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

A representative from Paterson should be on-site periodically to observe placement of the fill and excavated native soils and to conduct compaction testing on each lift of fill placed.

5.3 Foundation Design

Raft Foundation

For support of the proposed multi-storey buildings, consideration should be given to using a raft foundation due to the expected building loads. For four levels of underground parking, it is anticipated that the excavation will extend to a depth such that the underside of the raft slab would be placed between geodetic elevations of 58 to 59 m.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The bearing resistance value at SLS (contact pressure) of **260 kPa** can be used for design purposes.

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 stress relief associated with the soil removal associated with four underground parking levels. The factored bearing resistance (contact pressure) at ULS can be taken as **390 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

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.

Based on four underground parking levels it is expected that the raft foundation will be installed on the silty clay deposit. The modulus of subgrade reaction was calculated to be **10 MPa/m** for a contact pressure of **260 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Deep Foundation – End Bearing Piles

A deep foundation method, such as end bearing piles, may be considered for the foundation support of the proposed building. 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 2. 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 (2) to four (4) piles would be recommended. 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. A full-time field review program should be conducted by Paterson field personnel during the pile driving operations to record the pile lengths, ensure that the refusal criteria is met and that piles are driven within the location tolerances (within 75 mm of proper location and within 2% of vertical). Paterson may undertake dynamic pile testing at the tome of pile driving and planning.

Table 2 - 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	1,100	9	27
245	11	1,050	1,250	9	31
245	13	1,200	1,400	9	35

The minimum recommended centre-to-centre pile spacing is 3 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock

End-Bearing Caisson Foundations

For the building support, a caisson foundation could also be utilized. It is expected that the caissons could either end bearing on the bedrock surface or socketed into the bedrock.

For end bearing caissons, the bedrock surface should be free of deleterious materials, loose soils and approved by the geotechnical consultant. The caissons can be constructed by advancing casing through the overburden soil to the bedrock surface (by vibrator or augering in advance of the casing), and seating the casing in the bedrock. For end-bearing caissons, the following bearing resistance value can be used:

❑ ULS value of **2,000 kPa** can be used for clean sound shale bedrock. This value incorporates a geotechnical resistance value of 0.5.

The reinforcement for the caissons should be designed by the structural engineer. If caissons are to be left exposed during winter months, some form of frost protection will be required to prevent frost adhesion and jacking of the casings. Further guidelines can be provided on these measures at the time of construction, if required.

Rock Socketed Caisson Foundation

For socketed caissons, they can be constructed by advancing casing through the overburden soils to the bedrock surface (by vibrator or augering in advance of the casing), seating the casing in the bedrock and then continuing drilling to create a rock socket. Considering the expected difficulty in cleaning and verifying the cleanliness of the bases of the caissons, it is recommended that the capacity of the rock socketed caissons be based solely on side wall resistance or socket shear.

Based on this, a factored socket shear resistance at ULS value of **750 kPa/m** can be used for clean sound shale bedrock sockets extending up to 3 m below the clean, bedrock surface, free of significant fractures and voids. This value incorporates a geotechnical resistance value of 0.4.

It is recommended that the ratio of the length to diameter of the usable socket be at least 3 for the above-noted socket shear resistance values to be applicable. It is recommended that the specified concrete strength for the caissons be at least 35 MPa, in order that the socket shear values are not limited by the concrete strength.

The deformation modulus, E_r, of the sound intact rock material can be taken to be about 400 times the unconfined compressive strength, or approximately 16,000 MPa. However, considering the bedding planes and other discontinuities, the deformation modulus, E_m, of the rock mass is expected to be closer to about 100 times the unconfined compressive strength, or approximately 4,000 MPa.

Foundation Uplift Resistance

Uplift forces on the proposed foundations can be resisted using the dead weight of the concrete foundations, the weight of the materials overlying the foundations, and the submerged weight of the caissons, where utilized. Unit weights of materials are provided in Table 3.

For soil above the groundwater level, calculate using the "drained" unit weight and below groundwater level use the "effective" unit weight. Backfilled excavations in low permeability soils can be expected to fill with water and the use of the effective unit weights would be prudent if drainage of the anchor footings is not provided.

As noted above, piles and caissons would be located below the groundwater level, so the submerged, or effective, weight of the foundation will be available to contribute to the uplift resistance, if required. Considering that this is a reliable uplift resistance, and is really counteracting a dead load, it is our opinion that a resistance factor of 0.9 is applicable for the ULS weight component.

Should the foundation uplift resistance capacities be insufficient for the foundation uplift loads, rock anchors should be utilized. This is discussed further in Section 5.7. A sieve analysis and standard Proctor test should be completed on each of the fill materials proposed to obtain an accurate soil density to be expected, so the applicable unit weights can be estimated.

Table 3 - Geotechnical Parameters for Uplift and Lateral Resistance Design

Material Description	Unit Weight (kN/m³)		Internal Friction Angle () φ'	Friction Factor, tan δ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_A	At-Rest K_O	Passive K_P
OPSS Granular A (Crushed Stone)	22.0	13.7	38	0.60	0.22	0.36	8.8
OPSS Granular B, Type II (Well-Graded Sand-Gravel)	21.5	13.4	36	0.55	0.26	0.41	7.5
In Situ Silty Clay	17.0	10.0	33	0.40	0.30	0.45	3.4

Notes:
- ❑ Properties for fill materials are for condition of 98% of standard Proctor maximum dry density.
- ❑ The earth pressure coefficients provided are for horizontal backfill profile.
- ❑ Passive pressure coefficients incorporate wall friction of 0.5 ϕ'.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a stiff silty clay bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V, passes only through in situ soil or engineered fill of the same or higher capacity as the soil.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building in accordance with the Ontario Building Code (OBC) 2012 and 2024. The results of the shear wave velocity testing are attached to the present report.

Field Program

The seismic array location is presented on Drawing PG4985-1 - Test Hole Location Plan presented in Appendix 2. Paterson field personnel placed 24 horizontal geophones in a straight line in a roughly north-south orientation. The 4.5 Hz. horizontal geophones were mounted to the surface by means of a 75 mm ground spike attached to the geophone land case. The geophones were spaced at 2 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12-pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at the center of the geophone array and 2, 3 and 23.5 m away from the last geophone and 2, 3 and 17.5 m from the first geophone.

Data Processing and Interpretation

Interpretation of the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile immediately below the proposed building foundations.

The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock shear wave velocity due to the increasing quality of bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on our testing results, the average overburden shear wave velocity is **189 m/s**, while the bedrock shear wave velocity is **2,210 m/s**. Should caissons or a raft foundation be founded at approximate elevation of 58 to 59 m, approximately 9.5 m of overburden will be present below the foundation.

The Vs_{30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and 2024 and as presented below:

$$V_{s30} = \frac{Depth_{of\ interest}(m)}{\left(\frac{Depth_{Layer1}(m)}{V_{s_{Layer1}}(m/s)} + \frac{Depth_{Layer2}(m)}{V_{s_{Layer2}}(m/s)}\right)}$$

$$V_{s30=} \frac{30\ m}{\left(\frac{9.5\ m}{189\ m/s} + \frac{20.5\ m}{2{,}210\ m/s}\right)}$$

$$V_{s30=}\ 506\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, Vs_{30}, for the proposed buildings bearing on piles or a raft slab foundation at an approximate geodetic elevation of 58 to 59 m is **506 m/s**. Therefore, a **Site Class C** is applicable for the proposed buildings, as per Table 4.1.8.4.A of the OBC 2012, or a **Site Designation X_{506}** as per the OBC 2024. The soil underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

It is expected that the basement area will be mostly parking, and a flexible or rigid pavement structure could be utilized.

Where a raft slab is utilized, a granular layer of OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

For buildings founded on piles or caissons, it is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

In consideration of the groundwater conditions encountered at the time of the construction, a sub-floor drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided in the clear stone under the lower basement floor.

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 structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 18 kN/m^3.

However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 11 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 the latest revision of the Ontario Building Code. 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 the latest revision of the Ontario Building Code.

5.7 Rock Anchor Design

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout from a 60 to 90-degree cone with the apex near the middle of the anchor bonded length. Interaction may develop between the failure cones of adjacent anchors resulting in a total group capacity less than the sum of the individual anchor load capacity.

A third failure mode of shear failure along the grout/steel interface should also be reviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada), have qualified personnel on staff to recommend appropriate rock anchor size and materials.

Centre-to-centre spacing between anchors should be at least four times the anchor hole diameter and greater than 1/5 of the total anchor length (minimum of 1.2 m) to lower the group influence effects. Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout does not flow from one hole to an adjacent empty one.

Regardless of whether an anchor is of the passive or post tensioned type, the anchor is recommended to be provided with a fixed length at the anchor base, which will provide the anchor capacity, and a free length between the rock surface and the bonded length. As the depth at which the apex of the shear failure cone develops midway along the bonded length, a fully bonded anchor has a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is at the bottom portion of the anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a sleeve to act as a bond break, with the sleeve filled with grout.

Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long-term performance of the foundation of the proposed buildings, the rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The unconfined compressive strength of shale bedrock ranges between 40 and 90 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. A **Rock Mass Rating (RMR) of 44** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.183 and 0.00009**, respectively. For design purposes, all rock anchors were assumed to be placed at least 1.2 m apart to reduce group anchor effects.

Recommended Grouted Rock Anchor Lengths

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented on the following page. Load specified rock anchor lengths can be provided, if required.

For our calculations the following parameters were used.

Table 4 - Parameters Used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Fair Quality Shale Hoek and Brown parameters	44 m=0.183 and s=0.00009
Unconfined compressive strength - Shale bedrock	40 MPa
Unit weight - Submerged Bedrock	15 kN/m3
Apex angle of failure cone	60
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75- and 125-mm diameter hole are provided in Table 5.

Table 5 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	3.0	1.5	4.5	250
	4.2	2.2	6.4	500
	6.5	2.6	9.1	1,000
	10	3.5	13.5	2,000
125	2.8	1.5	4.3	250
	3.5	2.4	5.9	500
	5.5	2.8	8.3	1,000
	8	3.8	11.8	2,000

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel and flushed clean with water prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor holes.

Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared. The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

5.8 Pavement Design

Asphalt pavement is not anticipated to be required at the subject site such that a rigid pavement design is provided for the lowest basement level. However, should a flexible pavement be considered for the project, the recommended flexible pavement structures shown in Tables 6 and 7 would be applicable.

Table 6 - Recommended Flexible Pavement Structure - Car Only Parking Areas

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

Table 7 - Recommended Flexible Pavement Structure - Access Lanes

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill.	

Table 8 – Recommended Rigid Pavement Structure – Lower Parking Level

Thickness (mm)	Material Description
Specified by Others	**Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill.	

To control cracking due to shrinking of the concrete slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete slab. The control joints are generally recommended to be spaced at approximately 24 to 36 times the slab thickness (for example, a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete slab.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be sub-excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the SPMDD with suitable vibratory equipment.

Figure 5 – Podium Deck to Foundation Wall Drainage System Tie-In Detail which identifies the recommended transition from the podium deck pavement structure to an overburden pavement structure that would be located beyond the footprint of the subject structure. In summary, it is recommended to increase the thickness of the subbase layer for the portion of the pavement structure directly adjacent to the foundation structure. The subbase thickness would be tapered back to the pavement structure recommended herein at a 5H:1V taper extending upwards from the thickened subbase layer.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

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

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

Groundwater Suppression System Overview

It is recommended that a groundwater suppression system be provided for the proposed structures. It is expected that insufficient room will be available for exterior backfill and the foundation wall will be cast as a blind-sided pour against the shoring system.

It is understood that the proposed development will be constructed in two phases, and that Phase 2 of the development may not be undertaken until the completion of Phase 1. Paterson understands that the foundation wall located between the phases will be constructed using a double-sided methodology and that the excavation will be undertaken as an open-cut temporary excavation.

The following recommendations provide recommendations for implementing the proposed groundwater suppression system considering the temporary backfill condition along the phase-splitting foundation wall.

Groundwater Suppression System – Exterior Foundation Walls

It is recommended that the groundwater suppression system for the portions of the foundation wall blind-poured onto a temporary shoring system be undertaken as follows:

- ❑ A waterproofing membrane should be placed against the shoring system between underside of footings and 1 m below existing ground surface. The height of the waterproofing layer is recommended to extend up to a geodetic elevation of 70 m. The membrane is recommended to overlap below the overlying perimeter foundation footprint by a minimum of 600 mm inwards towards the building footprint and from the face of the overlying foundation.

- A composite drainage membrane (CCW MiraDRAIN 2000 or Delta-Teraxx or equivalent) should be placed against the waterproofing membrane with the geotextile layer of the drainage board facing the waterproofing layer between finished ground surface and the top of the footing.

- The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board to produce a suitable shingling effect. All end laps of the drainage board sheets should overlap abutting boards by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by Paterson field personnel.

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

The top endlap of the foundation drainage board should be provided with a suitable termination bar against the foundation wall to mitigate the potential for water to perch between the drainage board and foundation wall. If a pile or caisson-style foundation is considered for the proposed structure, provisions should be carried to provide a minimum 75 mm thick mud slab for the perimeter strip footings/pile caps to provide an adequately smooth and prepared substrate for installing the horizontal portion of membrane terminating below the perimeter footing.

Reference should be made to Figure 2 – Water Suppression System in Appendix 2 of this report which depicts the above-noted system for additional clarity. The currently proposed detail is considered preliminary and should be revised once foundation design and perimeter foundation wall construction methodologies are known.

Groundwater Suppression System – Open-Cut Excavation Section

Since the foundation wall located along the phase line will be temporarily located below the local long-term groundwater table until the second phase basement level is constructed, a temporary groundwater suppression system is recommended to be implemented for this portion of the structure.

This is recommended to mitigate dewatering of the long-term groundwater table following the earthworks stage of the first phase and subsequent operation of the buildings foundation drainage system, and to mitigate the potential for surface water to infiltrate into the lower basement levels.

The following recommendation is provided on the basis that the second phase may be undertaken within 2 to 3 years upon completion of the first phase and is not considered an alternative permanent groundwater suppression mitigation solution to the above-noted design for the portions of the foundation wall located along the property boundary.

Consideration may be otherwise provided to implementing the above-noted groundwater suppression system (i.e., waterproofing membrane placed on geotextile face of composite foundation drainage board) for this portion of the foundation wall, however, it is not expected these materials will be able to be re-used along the perimeter foundation walls for the subsequent development phase.

It is recommended that a composite drainage be installed against the exterior foundation wall for the height of the foundation wall located between Phase 1 and Phase 2. The drainage membrane should be backfilled upon with a minimum 3 m thick layer of compacted, site-generated, workable and impermeable brown silty clay for the entire height and length of the foundation wall. It would be recommended that a waterproofing membrane be placed on the geotextile face of the foundation drainage board for a minimum of 4.8 m horizontally from the corner of the adjacent foundation wall/property boundary.

The Paterson-reviewed and -approved silty clay fill should be placed in dry and above-freezing conditions in maximum 300 mm thick lifts and adequately proof-rolled with a suitably sized vibratory sheepsfoot roller under Paterson's supervision. This backfill would be recommended to be provided between the base of the excavation and to a minimum geodetic elevation of 70 m and corresponding to the height of the waterproofing membrane. The remaining backfill may consist of either free-draining crushed stone or other approved material in conjunction with the foundation drainage layer.

Elevator Shaft and Additional Sub-Floor Structures Waterproofing

Elevator shafts located below the underslab drainage system should be provided full-depth positive-side waterproofing and provided with a PVC waterstop at the shaft wall and footing interface. Review of architectural design drawings should be completed by Paterson for the above-noted items once the building design has been finalized and prior to tender.

A positive-side (i.e., placed on exterior faces) waterproofing system should also be provided for any elevator shafts, pools, cisterns and other water-tight structures that will be located within the lowest basement level.

The 100 to 150 mm diameter perforated corrugated underfloor drainage pipe should be placed along the perimeter of the exterior sidewalls and provided a gravity connection to the sump pump basin. All portions of the above-noted assembly should be verified at the time of implementation by Paterson personnel.

Interior Perimeter and Underfloor Drainage

An interior perimeter and underfloor drainage system will be required to control water infiltration below the lowest underground parking level slab and redirect water from the building's foundation drainage system to the buildings sump pit(s). The interior perimeter and underfloor drainage pipe should consist of a 100 to 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided with tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves.

The spacing of the underfloor drainage system should be provided by Paterson once the foundation and column layout and sump system location(s) have been finalized and during the design phase of the project (i.e., prior to tender).

Review of Architectural and Waterproofing/Drainage System Designs

Since a groundwater suppression and underfloor drainage system designed by Paterson is recommended to be implemented at the subject site, Paterson should review and advise on the architectural design of these features during the design phase and prior to tender.

Foundation Backfill

Backfill against the exterior sides of the foundation walls above the groundwater suppression system 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 CCW MiraDRAIN 2000 or Delta-Teraxx, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Podium Deck Waterproofing Tie-In

Waterproofing layers for podium deck surfaces should overlap across and below the top end lap of the vertically installed composite foundation drainage board to mitigate the potential for water to migrate between the drainage board and foundation wall and as depicted in Figure 5 – Podium Deck to Foundation Wall Drainage System Tie-In Detail.

Two methodologies have been provided to identify recommended procedures for double-sided and blind-sided foundation wall/podium deck areas. The blind-sided methodology will consist of installing the top portion of the foundation waterproofing layer upon the lagging prior to pouring the foundation wall and podium deck. The waterproofing layer would be later folded and adhered onto the podium deck waterproofing once that surface is waterproofed.

6.2 Protection of Footings Pile Caps and Grade Beams Against Frost Action

Footings, pile caps and grade beams of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

The underground parking area should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Excavation Side Slopes

At this site, temporary shoring will be required to complete the required excavations. However, it is recommended that where sufficient room is available open-cut excavation in combination with temporary shoring can be used.

Excavation Side Slopes

The subsoil at this site is considered to be mainly a Type 2 or Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. 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 the groundwater level.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. Excavation side slopes around the building excavation should be protected from erosion by surface water and rainfall events by the use of secured tarpaulins spanning the length of the side slopes, or other means of erosion protection along their footprint. The tarps should be anchored with stakes embedded a minimum of 600 mm below existing grade at the top of the excavation and on a maximum spacing of 2 m centres.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress. It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by "cut and cover" methods and excavations will not be left 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 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.

For design purposes, the temporary system may consist of soldier pile and lagging system or interlocking steel sheet piling and/or secant piles where settlement sensitive structures will be supported. Provisions will need to be carried to prepare these surfaces for the application of the exterior foundation waterproofing and drainage layers. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored, or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability.

The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

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

Table 9 – Soils Parameter for Shoring System Design

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_O)	0.5
Unit Weight (γ), kN/m^3	18
Submerged Unit Weight (γ), kN/m^3	11

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

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

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

Underpinning

Founding conditions of adjacent structures bordering the proposed building locations should be assessed and underpinning requirements should be evaluated based on proximity to the temporary excavation footprint.

6.4 Pipe Bedding and Backfill

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the SPMDD. The bedding material should extend at least to the spring line of the pipe.
The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the SPMDD.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD.

Clay seals are recommended to be implemented within the service trenches for site service connections to municipal sewers located with the City of Ottawa right-of-way. 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 compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

Groundwater Control for Building Construction

Due to the relatively impervious nature of the silty clay and existing groundwater level, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

A temporary Ministry of Environment, Conservation and Parks (MECP) Category 3 Permit to Take Water (PTTW) may be required if more than 400,000 L/day are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

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.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building's perimeter or sub-slab drainage system will be directed to the proposed building's sump pit. It is expected that groundwater flow will be low (i.e.- less than 15,000 L/day) with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps. Confirmation of the actual groundwater flow should be completed by the geotechnical consultant at the time of construction.

Impacts on Neighboring Properties

It is understood that four levels of underground parking are planned for the proposed buildings. Based on the existing groundwater level and considering the proposed building will be surrounded by a waterproofing membrane, long-term groundwater lowering will be minimal and take place within a limited range of the proposed buildings. Based on the proximity of neighboring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighboring structures.

As a precautionary measure, a **water-tight shoring system** comprising of "interlock sheet piles" or "secant piles", with sufficient embedment depth below the excavation level is recommended to be installed along the east property boundary. The proposed water-tight shoring system and increased embedment depth of the wall below the final excavation level will control any groundwater flow into the excavation, which would pass through the underlying low permeability clay soil and will consequently reduce the amount of dewatering that occurs.

It should be noted that no issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed buildings due to the proposed water suppression system, which will limit dewatering over long-term conditions.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil.

Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable. In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and tarpaulins or other suitable means.

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

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

Under winter conditions, if snow and ice is present within the blast rock or other imported fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized in settlement-sensitive areas.

6.7 Landscaping Considerations

Tree Planting Considerations

It is understood the proposed building will include four levels of underground parking and the structures will be founded at a minimum of 13 m below finished grade. Given the depth of foundations proposed for the structure, it is expected that the support of the foundations derives from soil located below the depth that could be impacted by tree roots for trees located within the area of the subject site.

Therefore, foundation distress due to potential moisture depletion caused by trees is not expected to be experienced by the proposed structure. Since the proposed structure is not anticipated to be founded upon silty clay soils affected by the depth of root penetration, City approved trees within the subject site will not be subject to planting restrictions as based on the *City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines*) from a geotechnical perspective.

7.0 Recommendations

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

- Review of the proposed structure(s) and adjacent structures from a geotechnical perspective.
- Field review of the installation of the drainage and waterproofing systems from a geotechnical perspective.
- Review of the caisson operations during implementation, if applicable.
- Review of underfloor drainage system layout.
- Review of waterproofing of building's elevator pit and sump pit.
- Review of the water suppression system installed against the foundation wall along Phase 1 and Phase 2 of the proposed development.
- Review of underpinning design for adjacent buildings, if required.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials used.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

All excess soils generated by construction activities should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. Also, our recommendations should be reviewed when the project drawings and specifications are complete.

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

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Taggart Realty Management 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.

LICENSED PROFESSIONAL ENGINEER
February 3, 2025
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

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Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - 267 O'Connor Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southeast corner of O'Connor Street and Gilmour Street. Geodetic elevation = 71.884m.

REMARKS

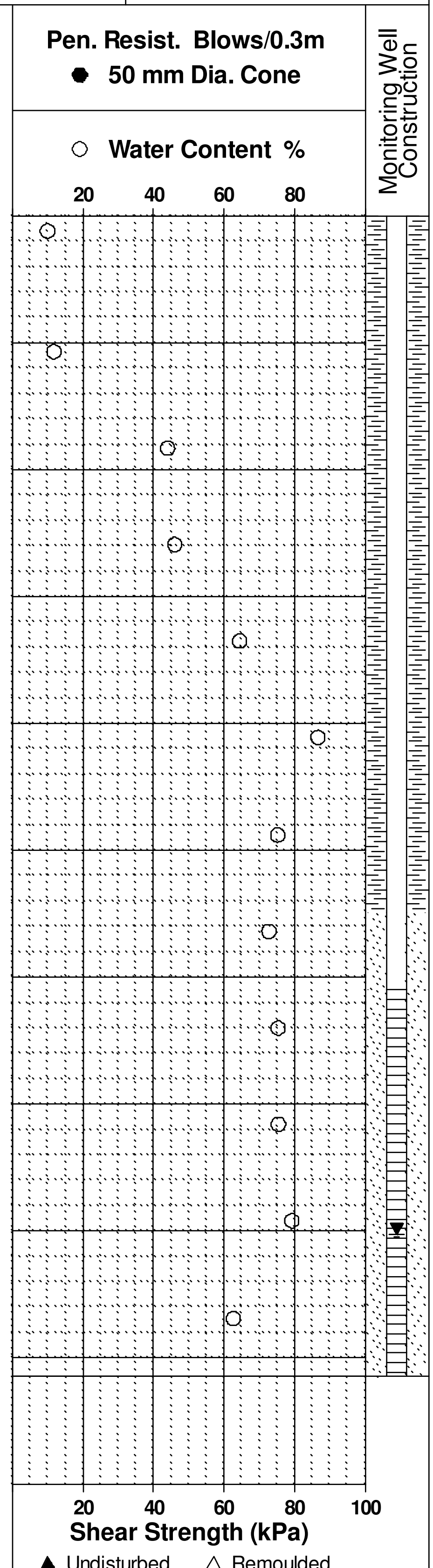
BORINGS BY CME 55 Power Auger

DATE March 22, 2014

FILE NO. PG3176

HOLE NO. BH 1-14

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	71.95
50mm Asphaltic concrete over crushed stone	0.25	AU	1				
FILL: Brown silty sand with gravel, concrete and brick		SS	2	100	50+	1	70.95
	1.83	SS	3	33	12	2	69.95
Stiff, grey-brown **SILTY CLAY**		SS	4	92	8		
- grey by 3.0m depth						3	68.95
		SS	5	100	7		
		SS	6	100	2	4	67.95
		SS	7	100	2	5	66.95
		SS	8	100	1	6	65.95
		SS	9	100	1		
		SS	10	100	W	7	64.95
		SS	11	100	1	8	63.95
		SS	12	100	W		
	9.14					9	62.95
End of Borehole							
(GWL @ 8.03m-April 2, 2014)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - 267 O'Connor Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southeast corner of O'Connor Street and Gilmour Street. Geodetic elevation = 71.884m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 22, 2014

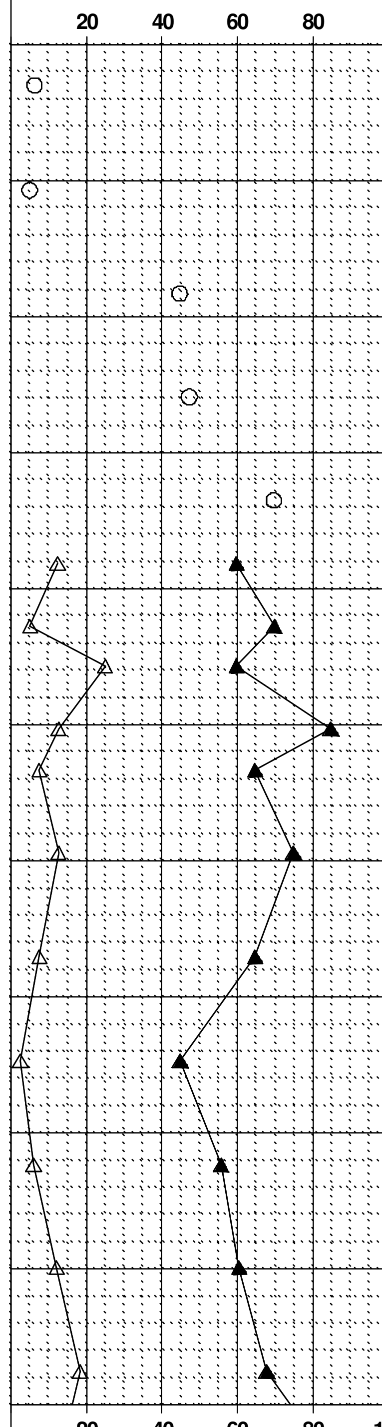
FILE NO. PG3176

HOLE NO. BH 2-14

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	71.71
75mm Asphaltic concrete over crushed stone 0.41		AU	1				
FILL: Brown silty sand 1.52		SS	2	50	15	1	70.71
Stiff, brown SILTY CLAY - grey by 3.0m depth		SS	3	92	5	2	69.71
		SS	4	88	4	3	68.71
		SS	5	100	1	4	67.71
						5	66.71
						6	65.71
						7	64.71
						8	63.71
						9	62.71
						10	61.71

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

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Monitoring Well Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - 267 O'Connor Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southeast corner of O'Connor Street and Gilmour Street. Geodetic elevation = 71.884m.

REMARKS

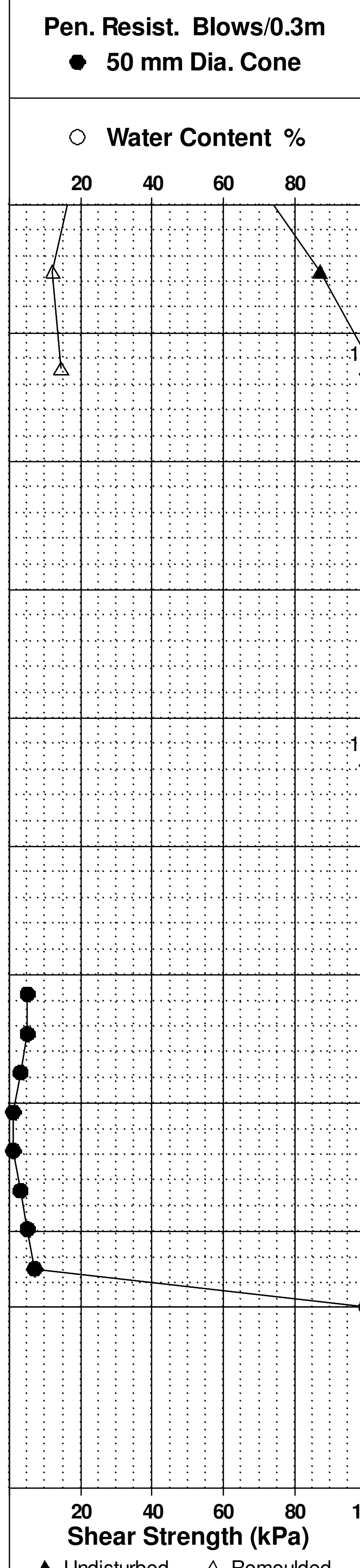
BORINGS BY CME 55 Power Auger

DATE March 22, 2014

FILE NO. PG3176

HOLE NO. BH 2-14

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
						10	61.71
						11	60.71
Stiff to very stiff, grey **SILTY CLAY**						12	59.71
						13	58.71
						14	57.71
14.33							
Dynamic Cone Penetration Test commenced at 14.33m depth.						15	56.71
						16	55.71
						17	54.71
						18	53.71
18.59							
End of Borehole Practical DCPT refusal at 18.59m depth.							



Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - 267 O'Connor Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southeast corner of O'Connor Street and Gilmour Street. Geodetic elevation = 71.884m.

REMARKS

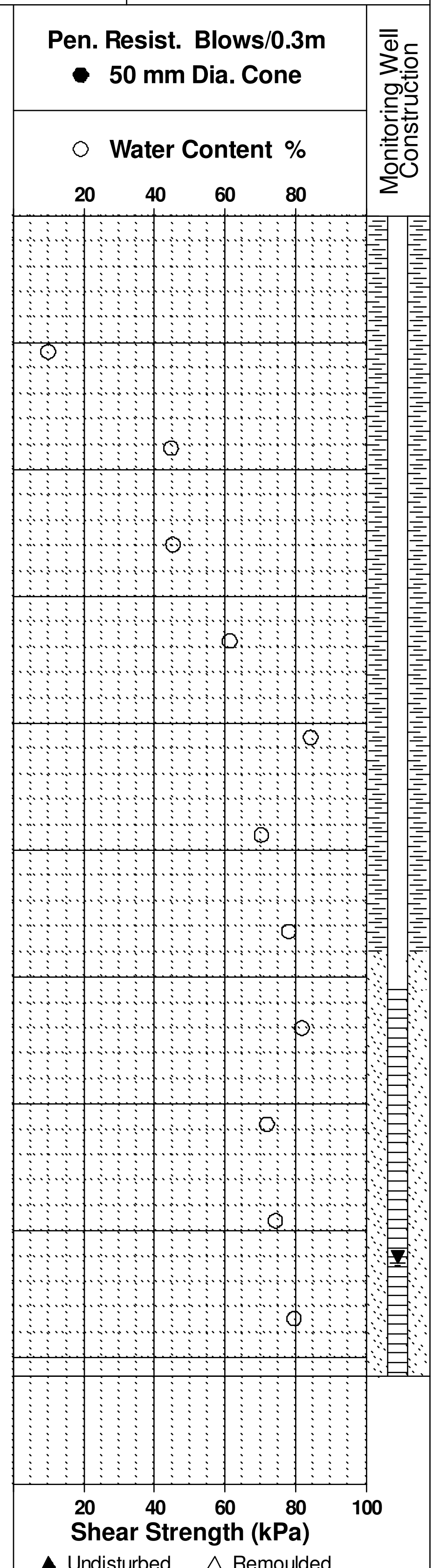
BORINGS BY CME 55 Power Auger

DATE March 22, 2014

FILE NO. PG3176

HOLE NO. BH 3-14

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	71.53
50mm Asphaltic concrete over crushed stone	0.20						
FILL: Black silty sand with brick pieces		SS	1	8	28	1	70.53
	1.83	SS	2	42		2	69.53
Stiff to firm, brown **SILTY CLAY**		SS	3	58	9	3	68.53
- grey by 3.5m depth		SS	4	83	4		
		SS	5	100	2	4	67.53
		SS	6	100	2	5	66.53
		SS	7	100	2	6	65.53
		SS	8	100	1		
		SS	9	100	W	7	64.53
		SS	10	42	W	8	63.53
		SS	11	100	W		
	9.14					9	62.53
End of Borehole							
(GWL @ 8.25m-April 2, 2014)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - 267 O'Connor Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southeast corner of O'Connor Street and Gilmour Street. Geodetic elevation = 71.884m.

REMARKS

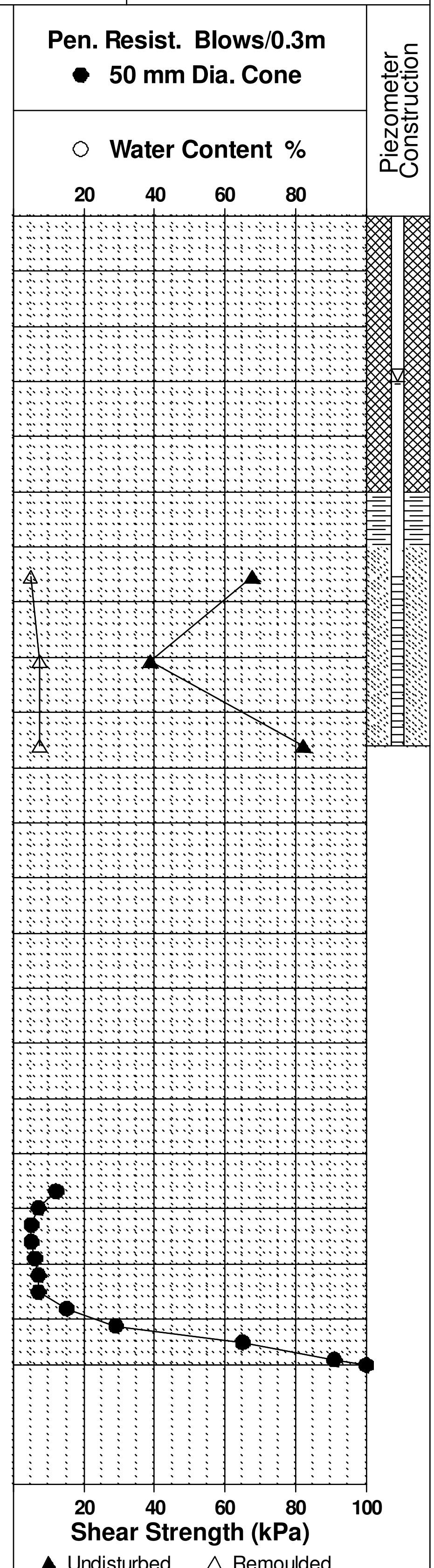
BORINGS BY CME 75 Power Auger

DATE November 24, 2012

FILE NO. PG3176

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	71.48
Asphaltic concrete 0.05							
FILL: Brown silty sand with gravel 1.12						1	70.48
Stiff, grey **SILTY CLAY**		SS	1	83	6	2	69.48
		SS	2	100	3	3	68.48
						4	67.48
		SS	3	100	2	5	66.48
						6	65.48
		SS	4	17	P	7	64.48
						8	63.48
9.60						9	62.48
Dynamic Cone Penetration Test commenced at 9.60m depth. Cone pushed to 17.4m depth.						10	61.48
						11	60.48
						12	59.48
						13	58.48
						14	57.48
						15	56.48
						16	55.48
						17	54.48
						18	53.48
(GWL @ 3.0m depth based on field observations)						19	52.48
20.83						20	51.48
End of Borehole Practical DCPT refusal at 20.83m depth.							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - 267 O'Connor Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southeast corner of O'Connor Street and Gilmour Street. Geodetic elevation = 71.884m.

REMARKS

BORINGS BY CME 75 Power Auger

DATE November 24, 2012

FILE NO. PG3176

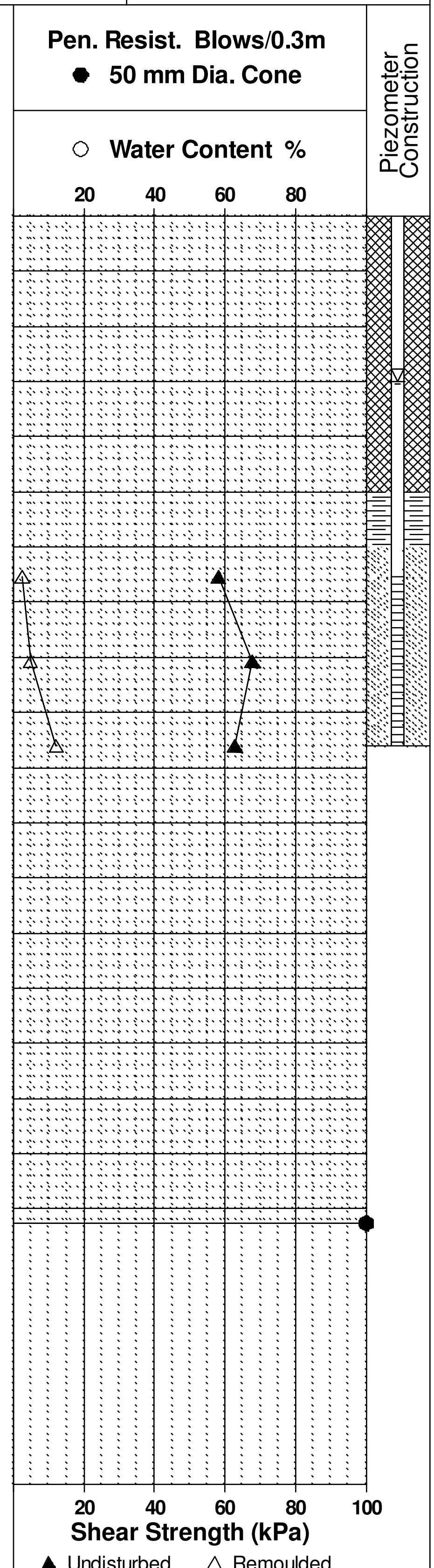
HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	71.40
Asphaltic concrete 0.05							
FILL: Brown silty sand with gravel and bricks 1.22						1	70.40
Stiff, grey **SILTY CLAY**		SS	1	83	5	2	69.40
		SS	2	75	3	3	68.40
						4	67.40
		SS	3	100	2	5	66.40
		SS	4	100	P	6	65.40
						7	64.40
						8	63.40
9.60						9	62.40
Dynamic Cone Penetration Test commenced at 9.60m depth. Cone pushed to 18.3m depth.						10	61.40
						11	60.40
						12	59.40
						13	58.40
						14	57.40
						15	56.40
						16	55.40
						17	54.40
18.26						18	53.40

End of Borehole

Practical DCPT refusal at 18.26m depth.

(GWL @ 3.0m depth based on field observations)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - 267 O'Connor Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located at the southeast corner of O'Connor Street and Gilmour Street. Geodetic elevation = 71.884m.

REMARKS

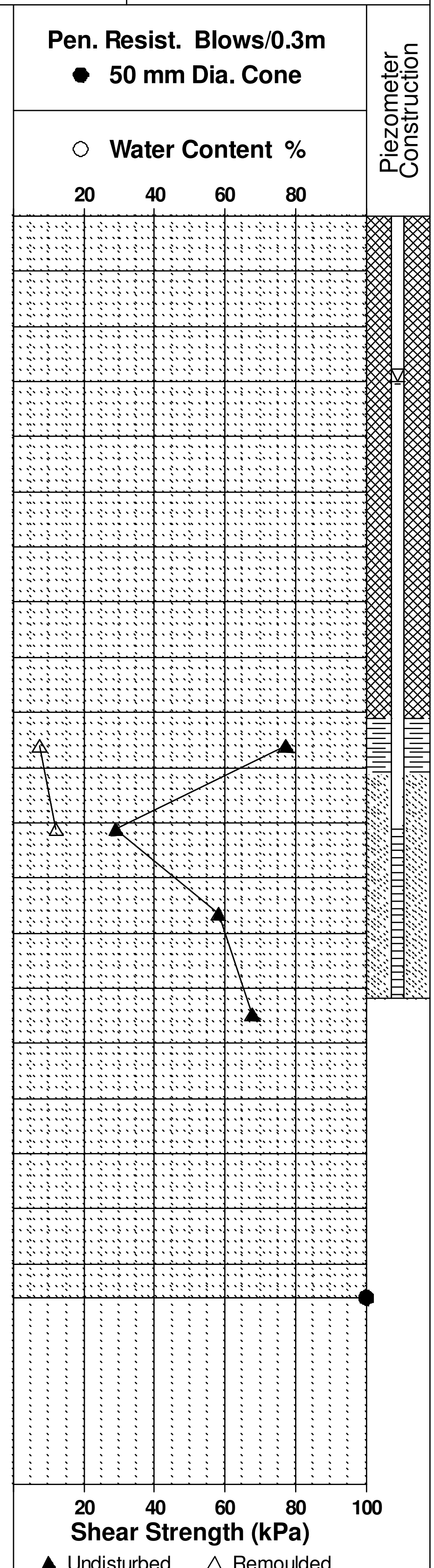
BORINGS BY CME 75 Power Auger

DATE November 24, 2012

FILE NO. PG3176

HOLE NO. BH 3

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	71.85
Asphaltic concrete 0.08						
FILL: Brown silty sand with gravel and bricks 1.37					1	70.85
	SS	1	33	9	2	69.85
	SS	2	83	4	3	68.85
					4	67.85
	SS	3	100	1	5	66.85
Stiff, grey **SILTY CLAY**					6	65.85
	SS	4	100	2	7	64.85
	SS	5	100	3	8	63.85
					9	62.85
	SS	6	75	P		
- firm to stiff by 10.7m depth					10	61.85
					11	60.85
					12	59.85
Stiff, grey **SILTY CLAY**					13	58.85
14.17					14	57.85
Dynamic Cone Penetration Test commenced at 14.17m depth. Cone pushed to 18.6m depth.					15	56.85
					16	55.85
					17	54.85
					18	53.85
					19	52.85
19.61						
End of Borehole Practical DCPT refusal at 19.61m depth. (GWL @ 3.0m depth based on field observations)						



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
Commercial Property - 267 O'Connor Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on the southeast corner of O'Connor Street and Gilmour Street. Geodetic elevation = 71.884m.

REMARKS

BORINGS BY Portable Drill

DATE August 6, 2020

FILE NO. PE4914

HOLE NO. BH 4-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Photo Ionization Detector ● Volatile Organic Rdg. (ppm) / ○ Lower Explosive Limit % (20, 40, 60, 80)	Monitoring Well Construction
GROUND SURFACE						0	68.60		
Floor tiles over concrete slab (to 0.91)									
Grey **SILTY CLAY**		SS	1	100		1	67.60		
		SS	2	92					
		SS	3	100		2	66.60		
		SS	4	100					
		SS	5	100		3	65.60		
(to 3.66)									
End of Borehole									
(GWL @ 0.12m - Aug. 13, 2020)									

100 200 300 400 500

RKI Eagle Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their "sensitivity". The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic Limit, % (water content above which soil behaves plastically)
PI	-	Plasticity Index, % (difference between LL and PL)
Dxx	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
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

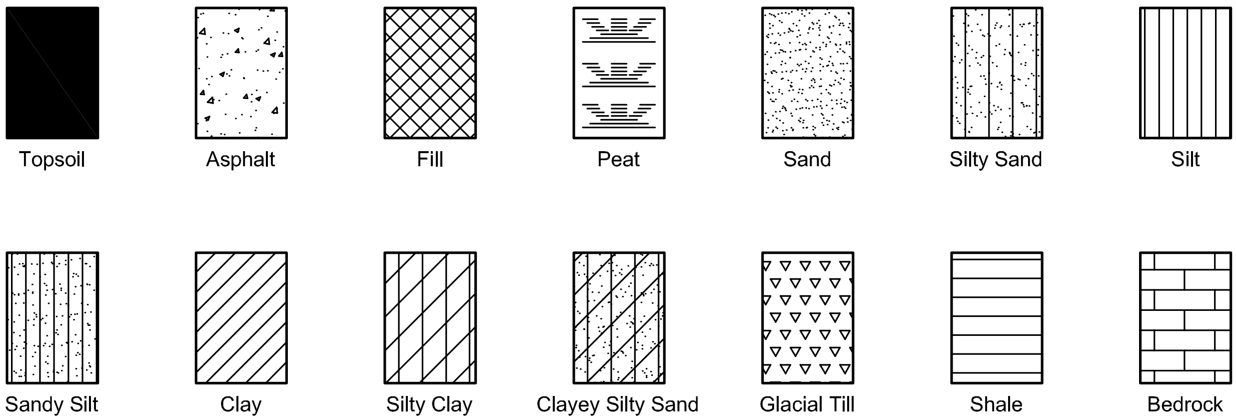
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

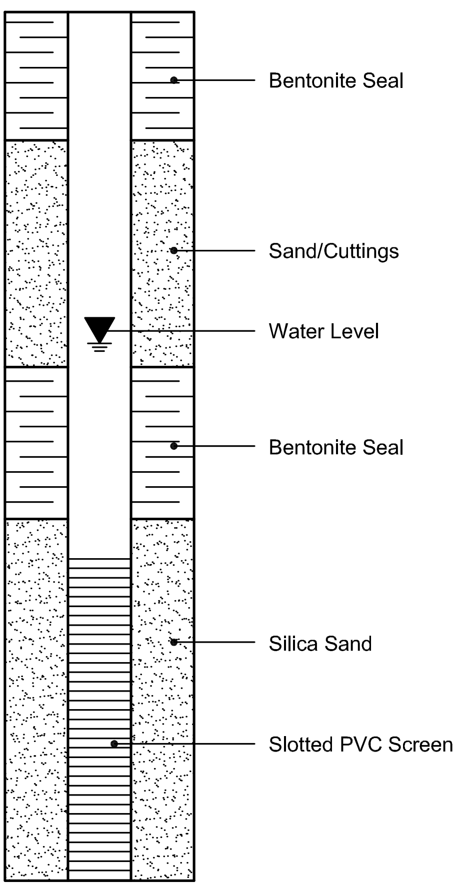
SYMBOLS AND TERMS (continued)

STRATA PLOT

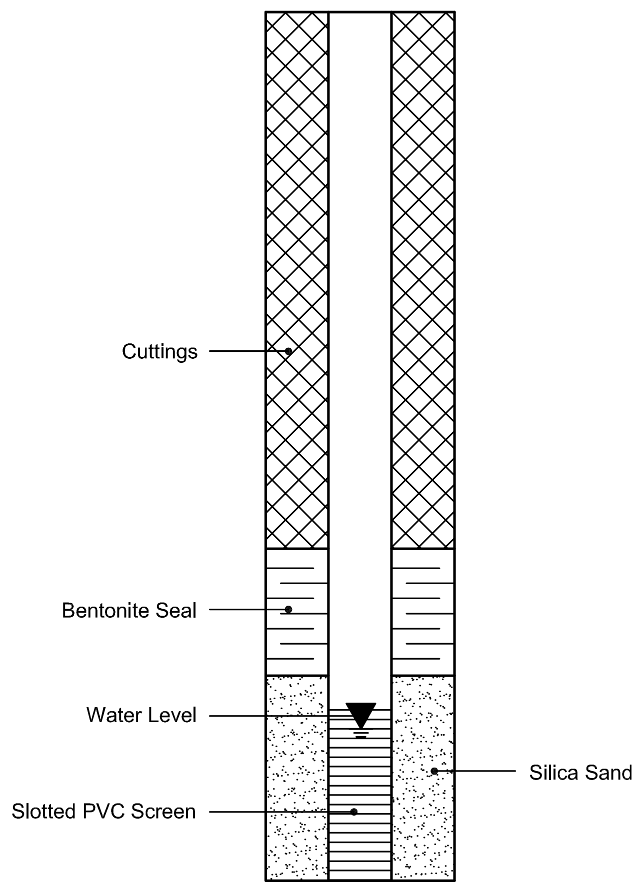


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 – WATER SUPPRESSSION SYSTEM

FIGURE 5 – PODIUM DECK TO FOUNDATION WALL DRAINAGAE SYSTEM TIE-IN DETAIL

FIGURES 6 & 7 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG4985-1 – TEST HOLE LOCATION PLAN

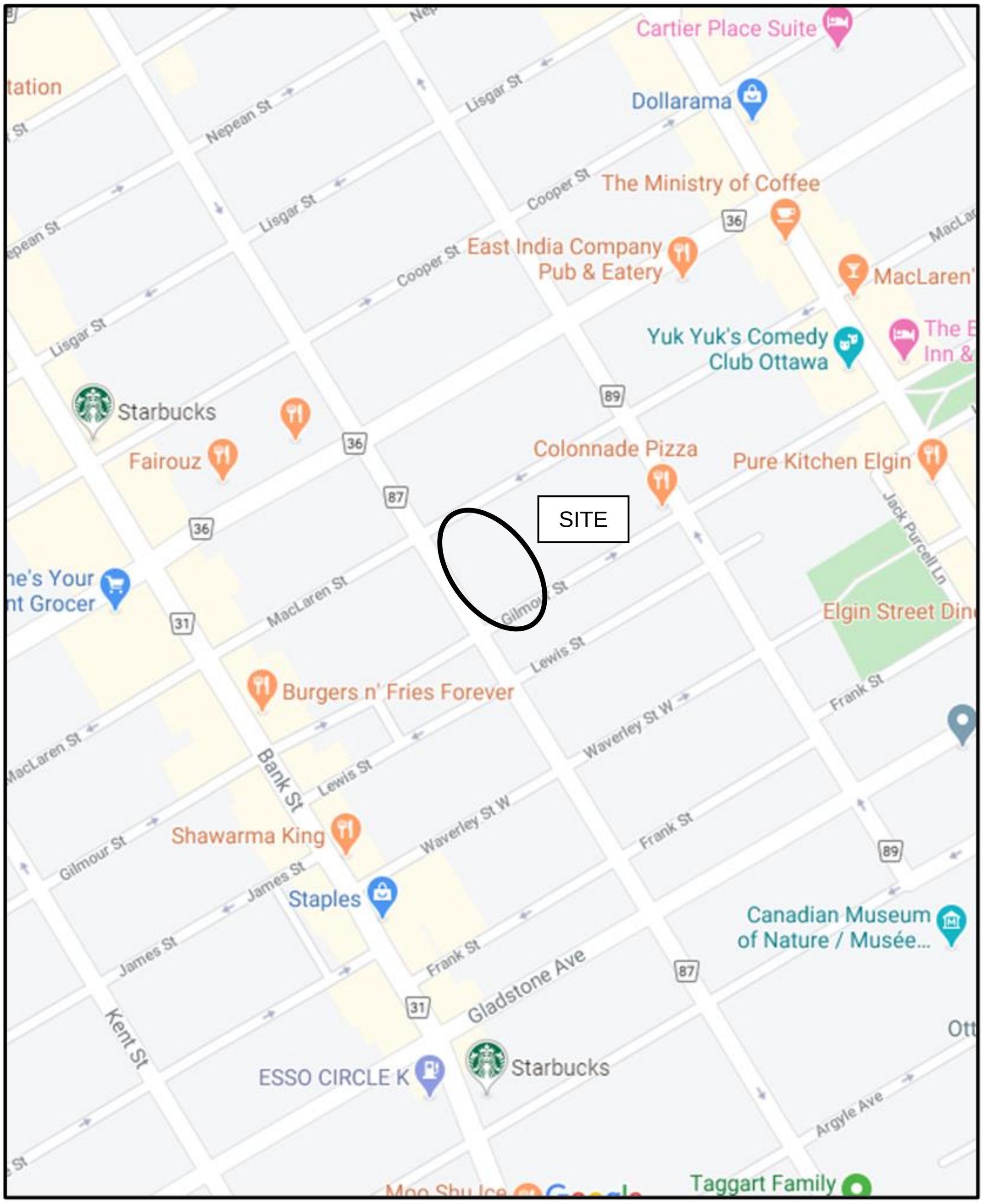
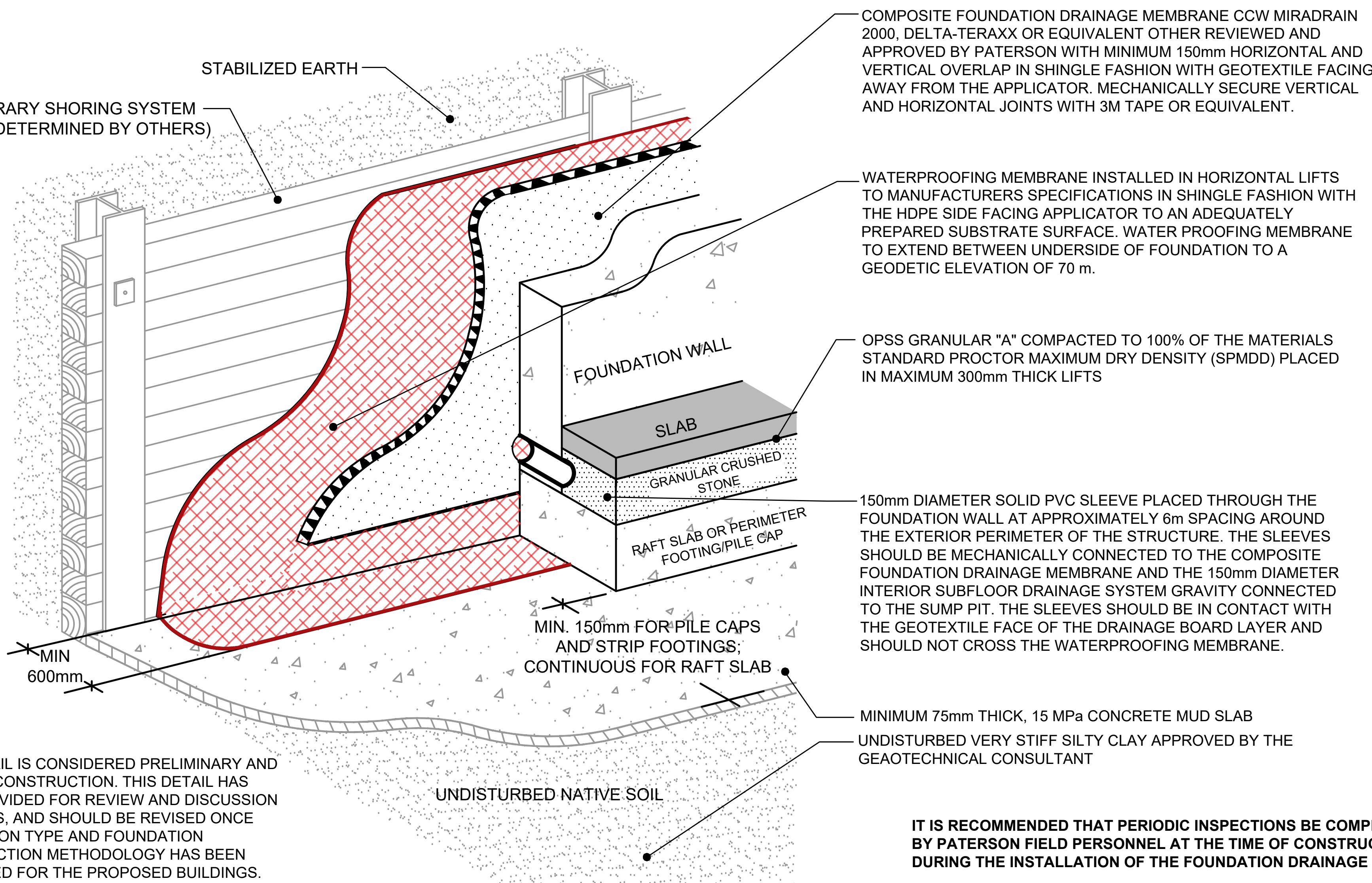


FIGURE 1

KEY PLAN

PATERSON GROUP



THIS DETAIL IS CONSIDERED PRELIMINARY AND NOT FOR CONSTRUCTION. THIS DETAIL HAS BEEN PROVIDED FOR REVIEW AND DISCUSSION PURPOSES, AND SHOULD BE REVISED ONCE FOUNDATION TYPE AND FOUNDATION CONSTRUCTION METHODOLOGY HAS BEEN CONFIRMED FOR THE PROPOSED BUILDINGS.

IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY PATERSON FIELD PERSONNEL AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE FOUNDATION DRAINAGE AREA.

PATERSON GROUP

TAGGART REALTY MANAGEMENT

PROPOSED HIGH-RISE BUILDINGS
267 O'CONNOR STREET

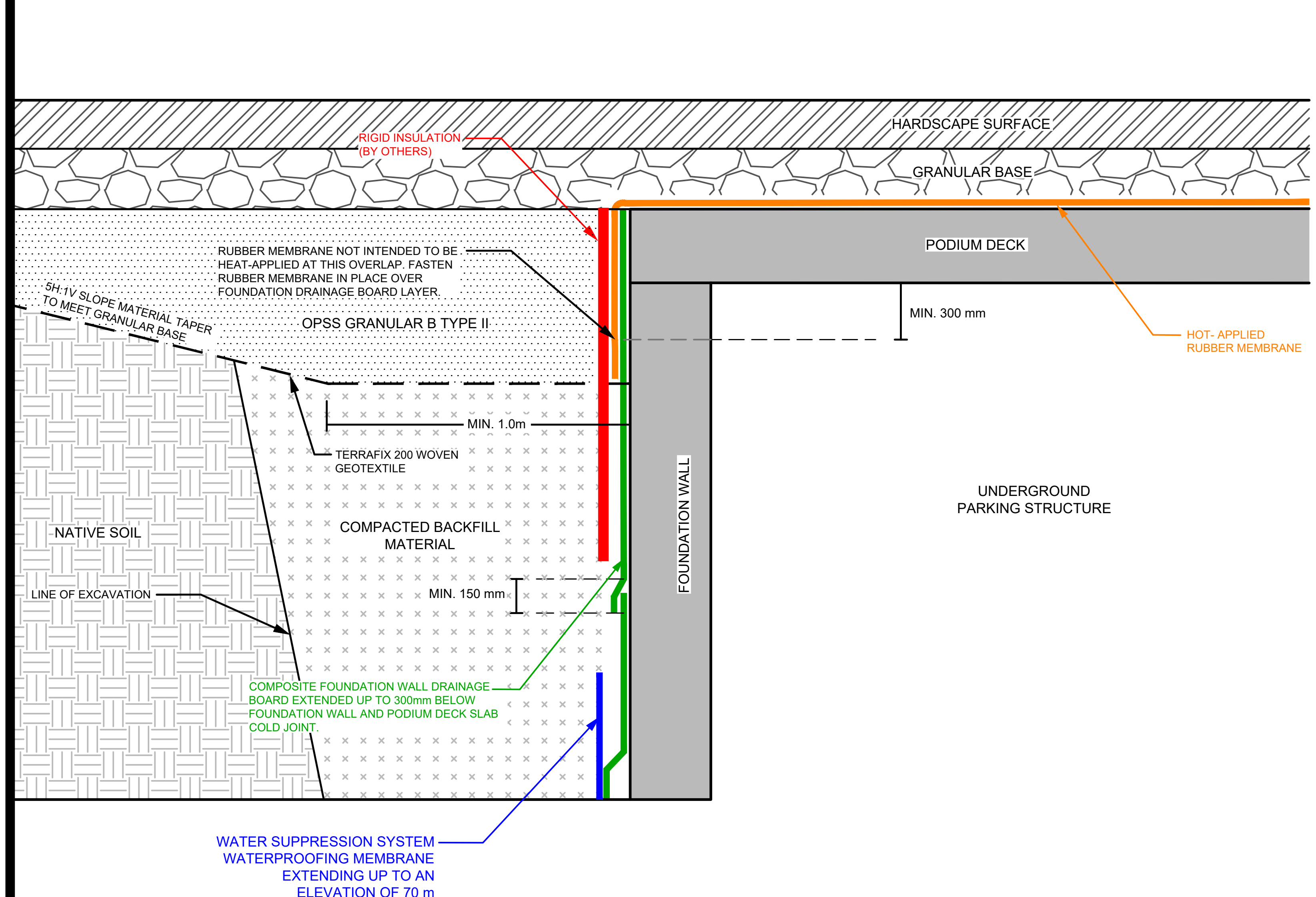
OTTAWA, ONTARIO

Title:

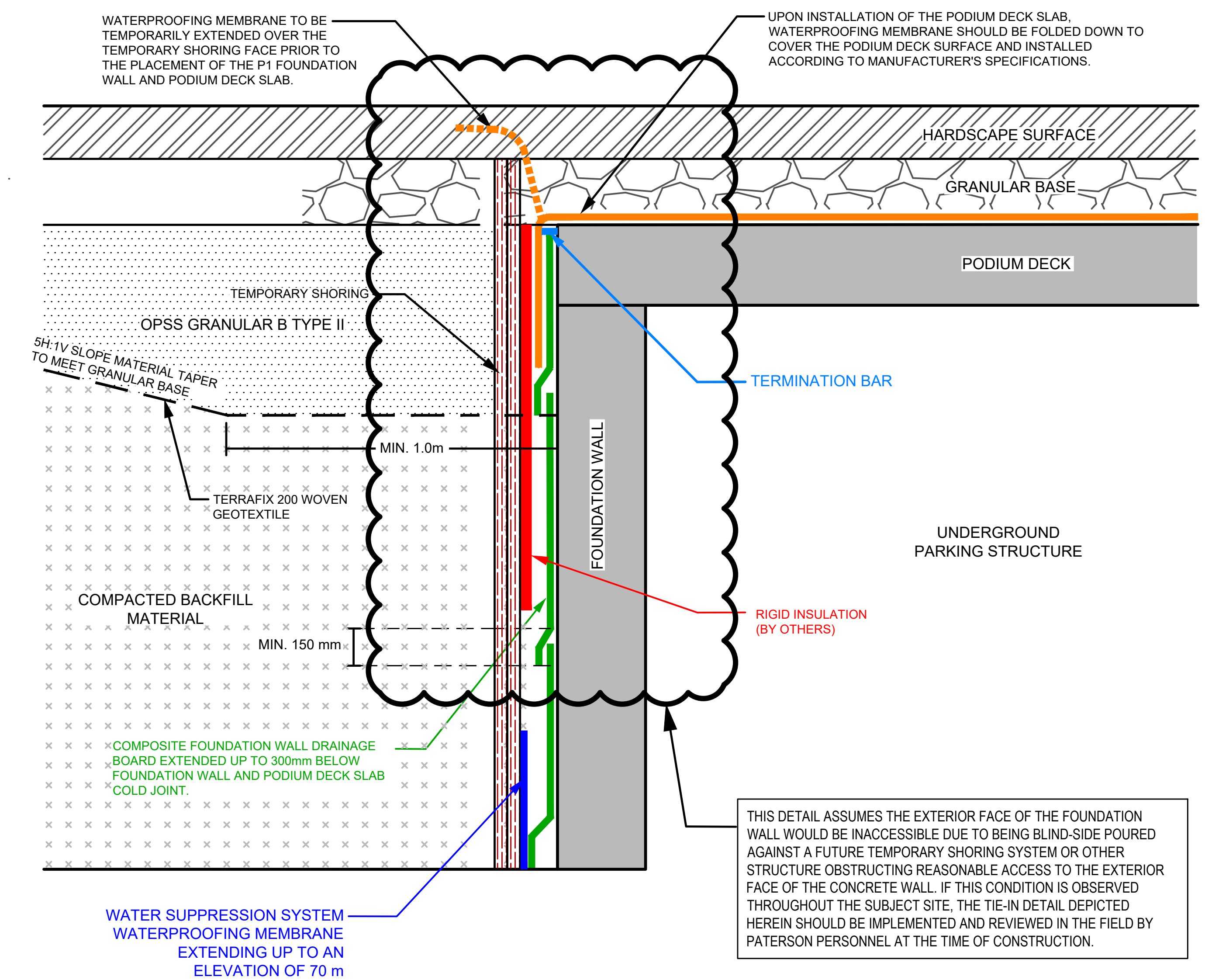
WATER SUPPRESSION SYSTEM

Scale: NTS	**Date:** 01/2025
Drawn by: NFRV	**Report No.:** PG4985-1
Checked by: DP	**Drawing No.:** FIGURE 2
Approved by: DG	**Revision No.:**

OPTION A - DOUBLE-SIDE POURED TOP OF FOUNDATION WALL



OPTION B - BLIND-SIDE POURED TOP OF FOUNDATION WALL



NOTES:

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

APPLICABILITY THICKNESS AND EXTENSIONS OF RIGID INSULATION ARE SPECIFIED BY OTHERS AND HAS BEEN DEPICTED HEREIN TO IDENTIFY PREFERRED LOCATION IN CONJUNCTION WITH WATERPROOFING AND DRAINAGE BOARD LAYERS IF SPECIFIED BY OTHERS.

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

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

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

NO.	REVISIONS	DATE	INITIAL
0			

TAGGART REALTY MANAGEMENT

GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE BUILDINGS
267 O'CONNOR STREET

OTTAWA, ONTARIO

Title: PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

Scale:	N.T.S	Date:	01/2025
Drawn by:	NFRV	Report No.:	PG4985-1
Checked by:	DP	Dwg. No.:	FIGURE 5
Approved by:	DG	Revision No.:	

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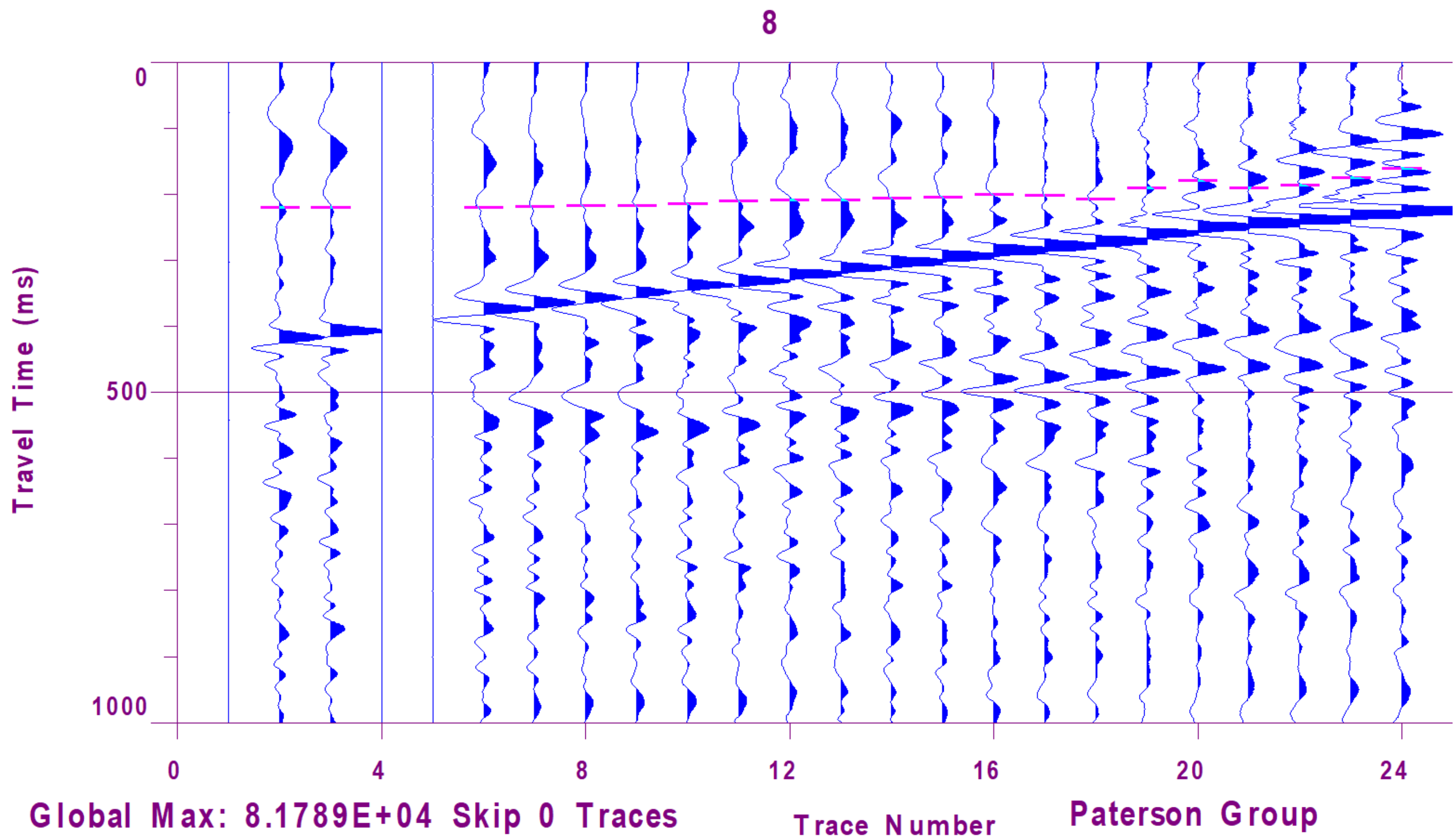


Figure 6 – Shear Wave Velocity Profile at Shot Location +69.5 m

PATERSON GROUP

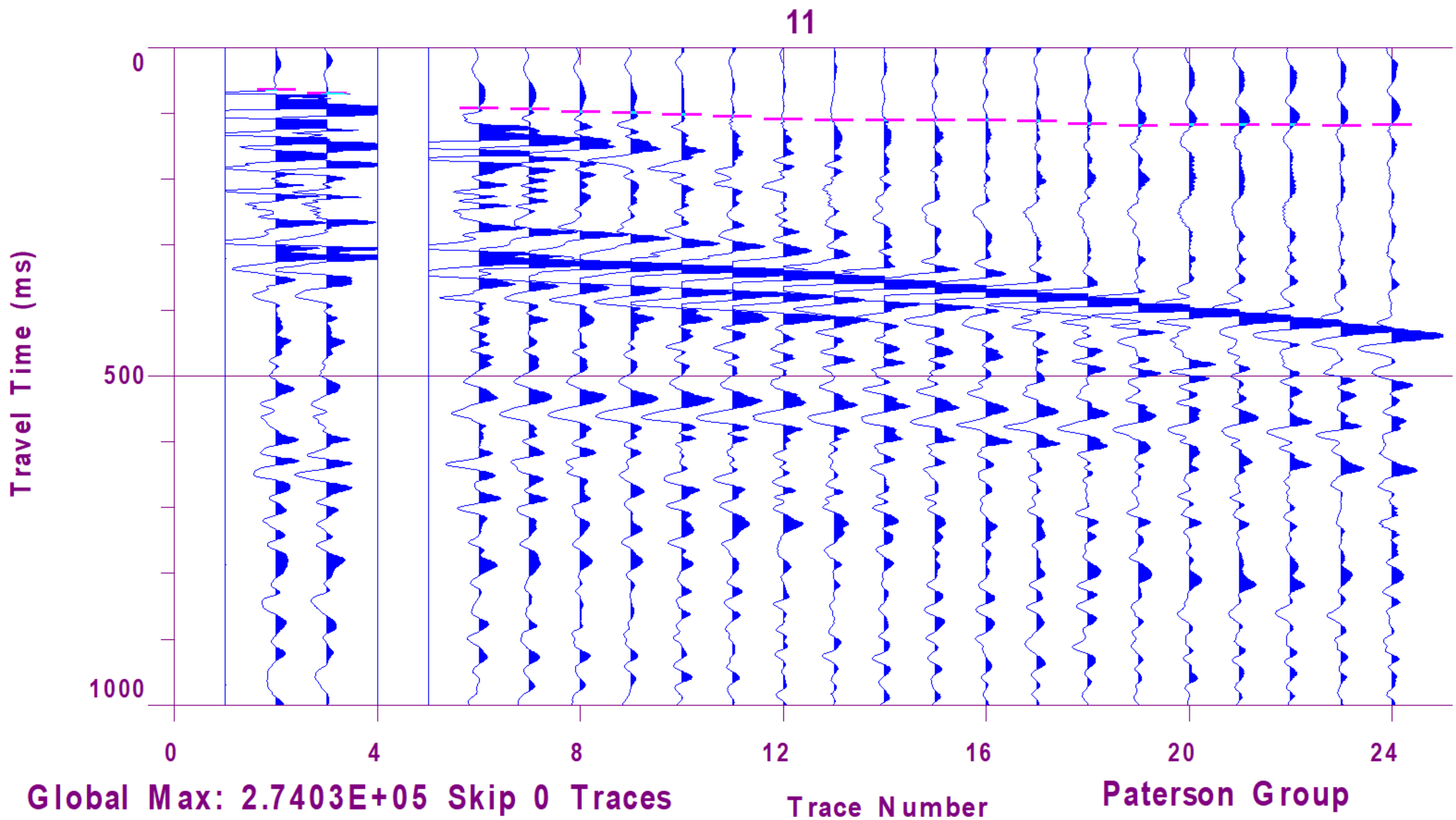
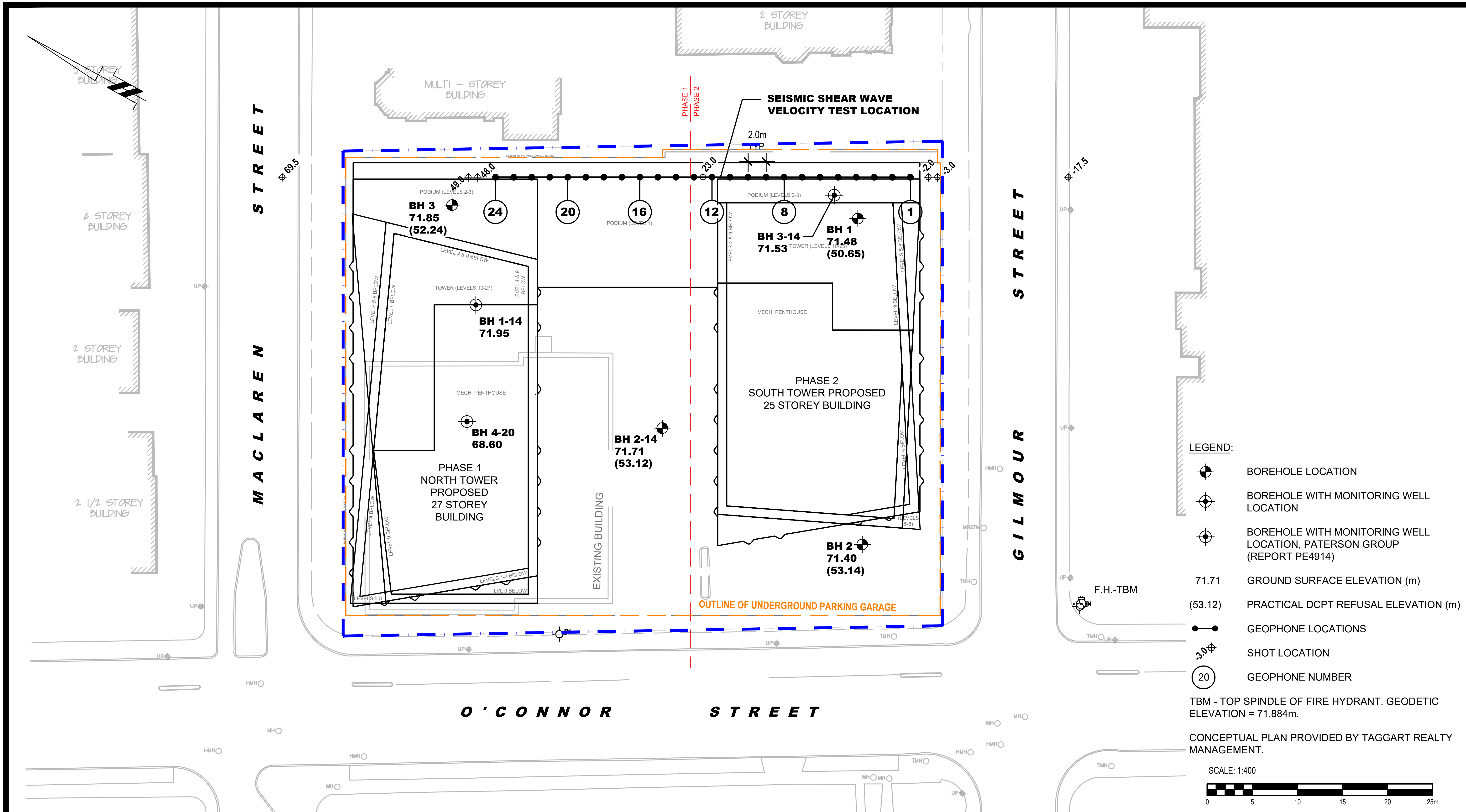


Figure 7 – Shear Wave Velocity Profile at Shot Location -2 m



PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL
3	AS PER REVISED CONCEPTUAL SITE PLAN	03/02/2025	NFRV
2	AS PER REVISED CONCEPTUAL SITE PLAN	16/01/2025	NFRV
1	SITE PLAN REVISED & SEISMIC SURVEY INFORMATION ADDED	24/09/2020	DP

TAGGART REALTY MANAGEMENT

GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE BUILDINGS - 267 O'CONNOR STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 03/2020
Drawn by: NFRV	Report No.: PG4985-1
Checked by: NFRV	Dwg. No.: PG4985-1
Approved by: DP	Revision No.: 3

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