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Geotechnical Investigation

Proposed High-Rise Buildings
267 O'Connor Street
Ottawa, Ontario

Prepared For

Taggart Realty Management

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Report: PG4985-1 Revision 1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Taggart Realty Management to prepare a geotechnical investigation report based on previous investigations completed by this firm for the proposed high-rise buildings to be located at 267 O'Connor Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

- ❑ Determine the subsoil and groundwater conditions at this site by means of boreholes.
- ❑ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes preliminary 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 preliminary geotechnical investigation.

2.0 Proposed Project

Based on the current conceptual plans, it is understood that a 28-storey (Phase 1) and 30-storey (Phase 2) residential buildings are proposed to be constructed at the subject site. Phase 1 is understood to occupy the southeast corner of the site while Phase 2 will occupy the footprint of the existing building along the north corner of the site. The existing building within the subject site is anticipated to be demolished as part of the proposed project. Also, 4 levels of shared underground parking are proposed to occupy the majority of the footprint of the subject site.

It should be noted that at-grade parking areas, landscaped areas and access lanes are anticipated as part of the proposed project. The proposed buildings are also anticipated to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the original 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 14.3m below existing grade. 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 rig 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 augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site, placed in sealed plastic bags, and transported to our laboratory for further review. The depths at which the split-spoon and auger samples 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 completed 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 all borehole 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 all boreholes to monitor the groundwater levels subsequent to the completion of the sampling program.

3.2 Field Survey

The borehole locations were determined by Paterson personnel taking into consideration the presence of underground and aboveground 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 soil investigation results.

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 Metcalfe 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 north 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 18.6m, 20.8 m, 18.3 m and 19.6 m depth below existing grade at BH2-14, 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 measurements were taken during the previous investigations and are presented on the Soil Profile and Test Data Sheets in Appendix 1. 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 be higher at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for construction of the proposed high-rise buildings. It is expected that the buildings will be founded over a raft foundation or deep foundations consisting of end bearing caissons extending to the bedrock or rock socketed caissons extending into the bedrock. Conventional end-bearing piles are not considered suitable for the proposed development due insufficient embedment depth available to provide sufficient end-fixity and lateral load resistance.

The caissons could also be utilized to provide foundation uplift resistance. However, should the caisson uplift resistance capacities, provided in Subsection 5.3, be insufficient for the foundation uplift loads, rock anchors should be utilized. The rock anchor design recommendations are discussed further in Subsection 5.7.

The above and other considerations are further 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 50 to 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 of 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.

Compacted Granular Fill Working Platform (Caisson Foundation)

Should the proposed high-rise building be supported on a caisson foundation, the use of heavy equipment would be required to install the caissons (i.e. driving crane). It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance.

A typical working platform could consist of 0.6 m of 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 caissons have been driven and cut off, the working platform can be regraded, and soil tracked in, or soil pumping up from the caisson 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.

Pressure Relief Chamber

A pressure relief chamber is recommended to be installed along with collection pipes within excavated 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 basement level within a utility room in close proximity to the proposed sump pit(s). Figure 3 - Pressure Relief Chamber in Appendix 2 provides an example of the required pressure relief chamber and Figure 4 - Temporary Drainage System Below Mudslab provides additional guidance on its implementation in conjunction with a mudslab. 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:

- Manage any water infiltration along the founding surface during the excavation program.
- Manage the water infiltration during the pouring of the raft slab to prevent water flow in the fresh concrete.
- Manage water infiltration below the raft slab until sufficient load is applied to resist any potential hydrostatic uplift.
- Regulate the discharge valve to control water infiltration once the raft slab 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 raft slab, the pressure relief valve will be fully closed to prevent any further dewatering.

Fill Placement

Fill used for grading beneath the proposed building, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill 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 building area should be compacted to at least 98% of the standard proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least 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.

For the caisson foundation option, it is expected that a granular working pad will be required and will have a minimum thickness of 600 mm. Alternatively, a 50 to 75 mm thick mudslab consisting of a minimum 15 MPa lean concrete.

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 the 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 **7 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.

End-Bearing Caisson Foundation

For the building support, a caisson foundation could also be utilized. It is expected that the caissons could either be end bearing on the bedrock surface or will be 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 the fractured nature of the upper layers of shale deposits and our experience with shale bedrock of the Billings formation throughout the Ottawa area the following socket shear resistance values should be considered for design:

- 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 useable 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 1.

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, caissons would be located below the groundwater level, so the submerged, or effective, weight of the caisson 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 caisson 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 1 - Geotechnical Parameters for Uplift and Lateral Resistance Design

Material Description	Unit Weight (kN/m^3)		Internal Friction Angle (°) φ'	Friction Factor, $\tan\delta$	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 Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012. 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 centre of the geophone array and 2, 3 and 23.5 m away from the first geophone and 2, 3 and 17.5 m from the last geophones.

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 as presented below:

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\left(\frac{(Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

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

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

Based on the results of the shear wave velocity testing, the average shear wave velocity, Vs_{30}, for the proposed buildings bearing on caissons 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. The soils 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 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 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 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 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 2 - 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/m^3
Apex angle of failure cone	60^o
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 3.

Table 3 - 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	1000
	10	3.5	13.5	2000
125	2.8	1.5	4.3	250
	3.5	2.4	5.9	500
	5.5	2.8	8.3	1000
	8	3.8	11.8	2000

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 Structure

Asphalt pavement is not anticipated to be required at the subject site. However, should a flexible pavement be considered for the project, the recommended flexible pavement structures shown in Tables 4 and 5 would be applicable.

Table 4 - 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 5 - 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	

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 98% of the SPMDD using suitable vibratory equipment.

6.0 Design and construction precautions

6.1 Foundation Drainage and Backfill

Water Suppression System

To manage and control groundwater infiltration over long term conditions, 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 for an illustration of this system cross-section):

- ❏ A waterproofing membrane will be required to lessen the effect of water infiltration for the underground parking levels starting at 3 m depth down to the foundation level. The waterproofing membrane will consist of bentonite panels such as Paraseal LG (20 mil HDPE Bentomat) fastened to the shoring system. The membrane should extend to the bottom of the excavation at the founding level of the proposed foundation.

- ❏ A composite drainage layer will be placed from finished grade to the bottom of the foundation wall. It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the bottom of the foundation wall. It is expected that 150 mm diameter sleeves placed at 3 m centres be cast in the foundation wall at the perimeter footing, grade beam, or raft slab 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 3 m which is above the vertical waterproofed area. The second source will be water from minor breaching of the waterproofing membrane.

Foundation Raft Slab Construction Joints

It is expected that the raft slab, where utilized, will be poured in sections. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Underfloor Drainage

Underfloor drainage will be required to control water infiltration below the lowest underground parking level slab. For design purposes, it is recommended that a 150 mm diameter perforated pipe be placed along the perimeter and in specific interior bays. 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.

Foundation Backfill

Where space is available, 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, as recommended above, 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.

Pressure Relief Chamber

The pressure relief chamber will be used to control the groundwater infiltration and hydrostatic pressure created by tanking the lower levels of underground parking. To avoid uplift on the raft foundation slab prior having sufficient loading to resist uplift, it is recommended that the water infiltration be pumped via the pressure relief chamber during construction.

The valve of the pressure relief chamber can be gradually close during construction 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. Figure 2 - Pressure Relief Chamber Detail in Appendix 2 provides a schematic of the recommended system.

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 may 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 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.

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 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.

For design purposes, the temporary system may 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 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 shoring system may be calculated using the following parameters.

Table 6 - Soil Parameters 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.

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. 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.

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 Neighbouring Structures

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 neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures. 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.

6.6 Winter Construction

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. The base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Precaution must be taken where excavations are carried out in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, 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.

7.0 Recommendations

The following material testing and observation program should be performed by a geotechnical consultant and is required for the foundation design data provided herein to be applicable:

- ❏ 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 geotechincal 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 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.

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
Sept.- 28, 2020
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- ❏ Taggart Realty Management
- ❏ Paterson Group

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

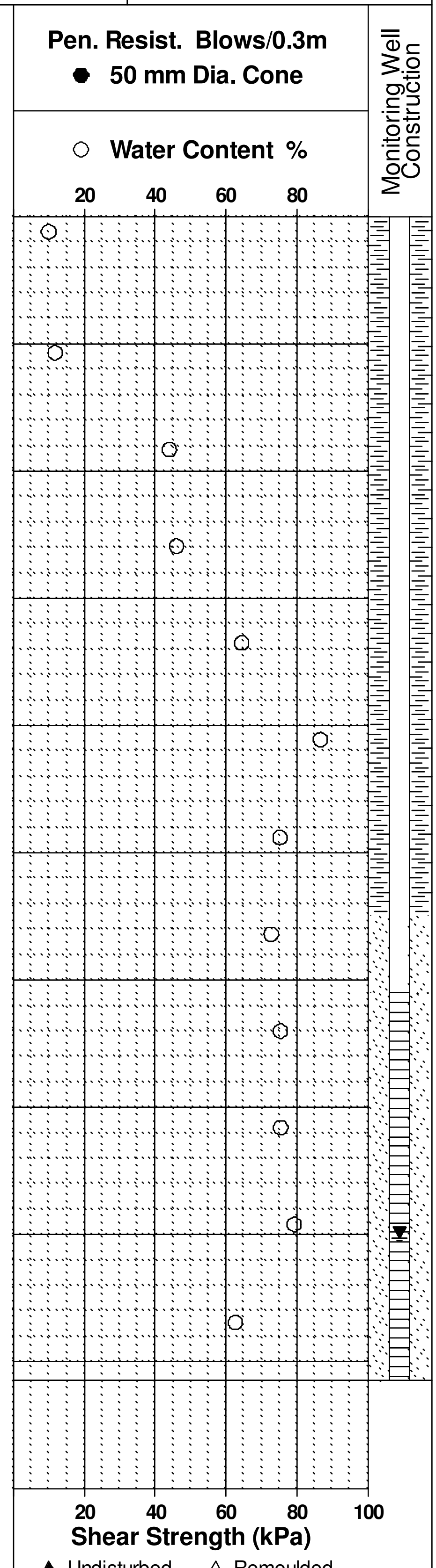
BORINGS BY CME 55 Power Auger

DATE March 22, 2014

FILE NO. PG3176

HOLE NO. BH 1-14

SOIL DESCRIPTION	Depth to base (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	3	68.95
- grey by 3.0m depth		SS	5	100	7	4	67.95
		SS	6	100	2		
		SS	7	100	2	5	66.95
		SS	8	100	1	6	65.95
		SS	9	100	1	7	64.95
		SS	10	100	W		
		SS	11	100	1	8	63.95
		SS	12	100	W		
End of Borehole	9.14					9	62.95
(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

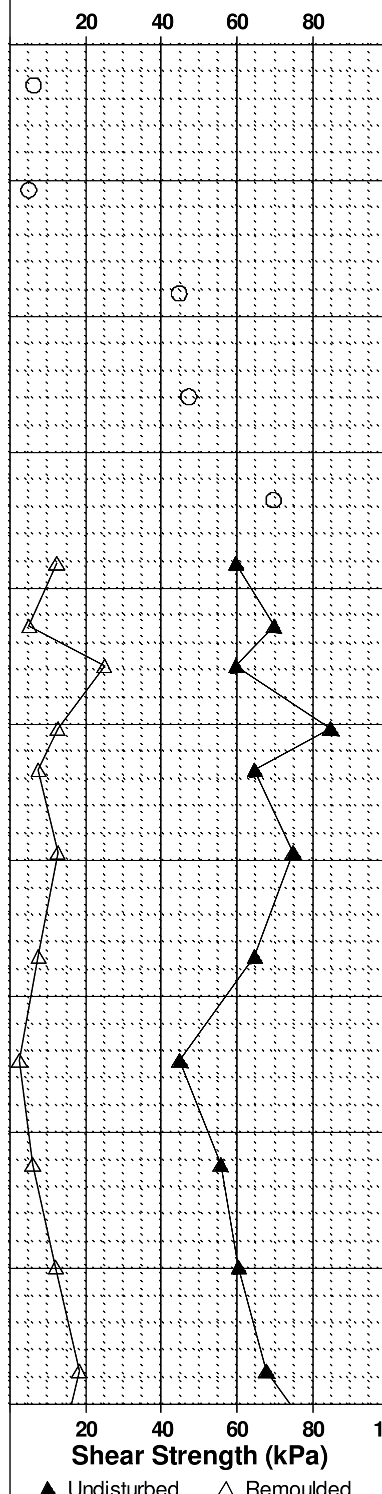
BORINGS BY CME 55 Power Auger

DATE March 22, 2014

FILE NO. PG3176

HOLE NO. BH 2-14

SOIL DESCRIPTION	TYPE	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



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

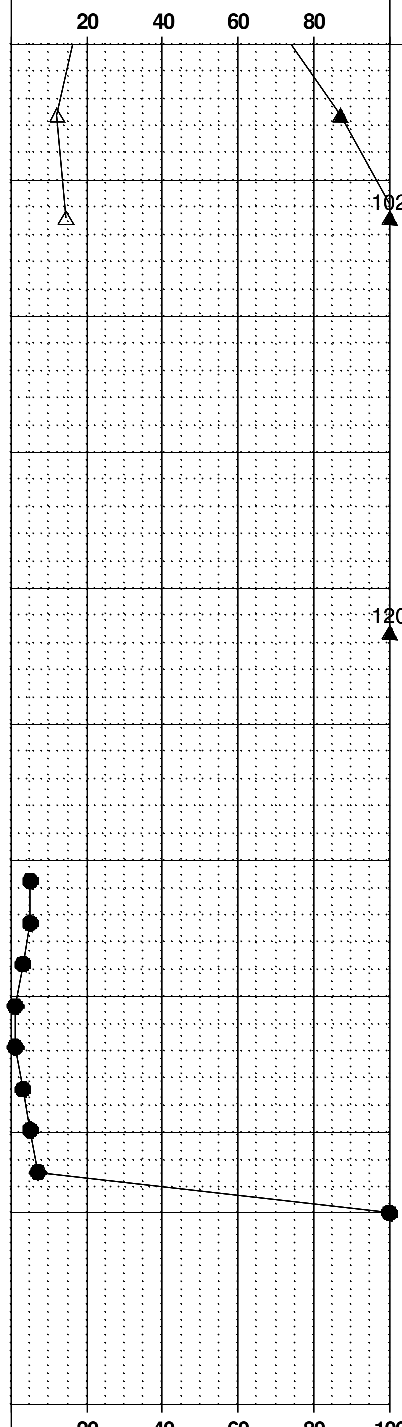
FILE NO. PG3176

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
14.33						14	57.71
Dynamic Cone Penetration Test commenced at 14.33m depth.						15	56.71
						16	55.71
						17	54.71
18.59						18	53.71
End of Borehole Practical DCPT refusal at 18.59m depth.							

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

○ Water Content %



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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

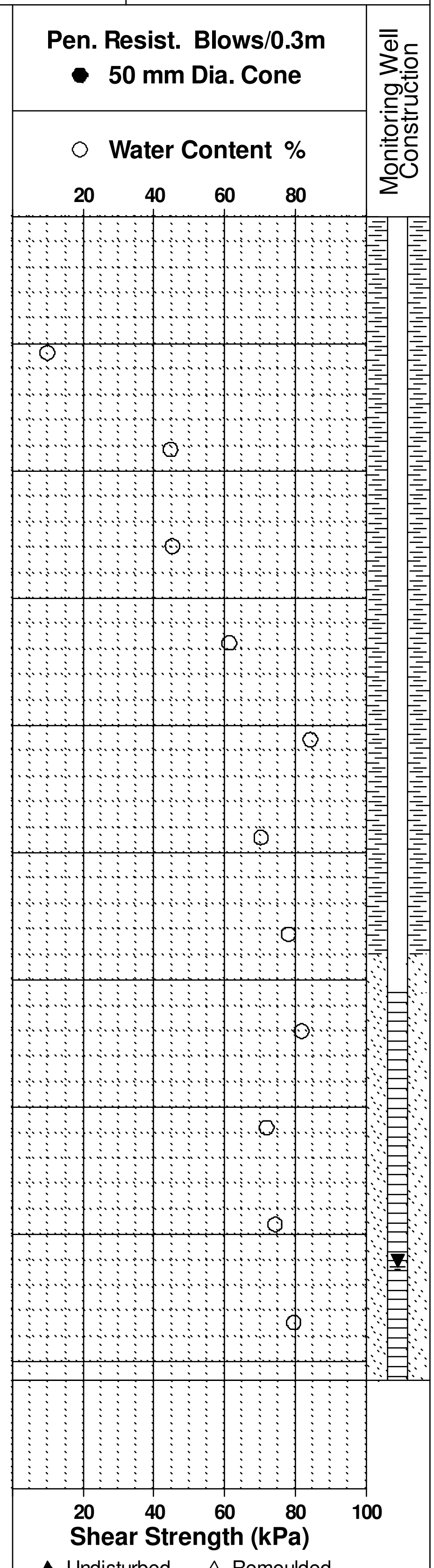
BORINGS BY CME 55 Power Auger

DATE March 22, 2014

FILE NO. PG3176

HOLE NO. BH 3-14

SOIL DESCRIPTION		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** - grey by 3.5m depth		SS	3	58	9	3	68.53
		SS	4	83	4	4	67.53
		SS	5	100	2	5	66.53
		SS	6	100	2	6	65.53
		SS	7	100	2	7	64.53
		SS	8	100	1	8	63.53
		SS	9	100	W	9	62.53
		SS	10	42	W		
	9.14	SS	11	100	W		
End of Borehole (GWL @ 8.25m-April 2, 2014)							



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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

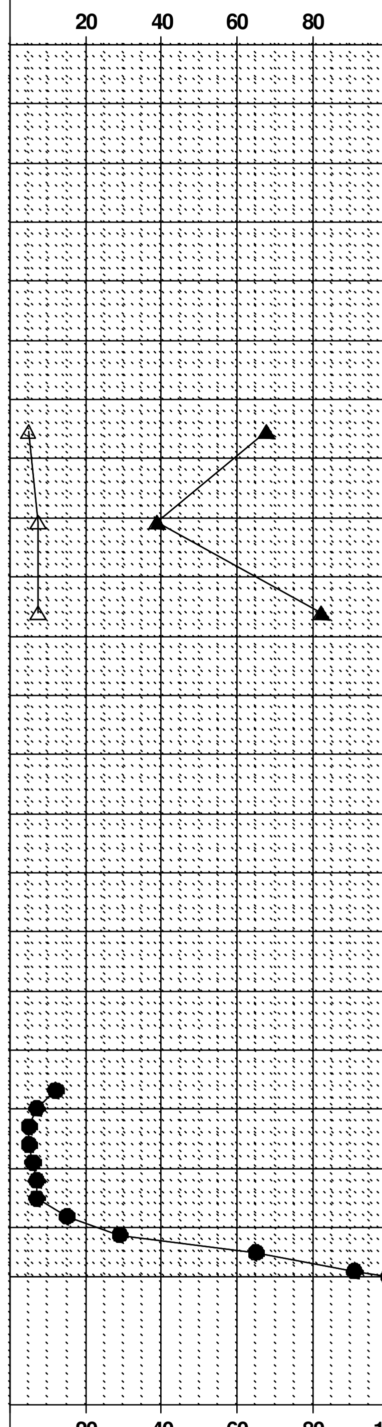
FILE NO. PG3176

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
Asphaltic concrete 0.05						0	71.48
FILL: Brown silty sand with gravel 1.12						1	70.48
		SS	1	83	6	2	69.48
		SS	2	100	3	3	68.48
						4	67.48
Stiff, grey **SILTY CLAY**		SS	3	100	2	5	66.48
		SS	4	17	P	6	65.48
						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.							

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

○ Water Content %



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

Piezometer 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 75 Power Auger

DATE November 24, 2012

FILE NO. PG3176

HOLE NO. BH 2

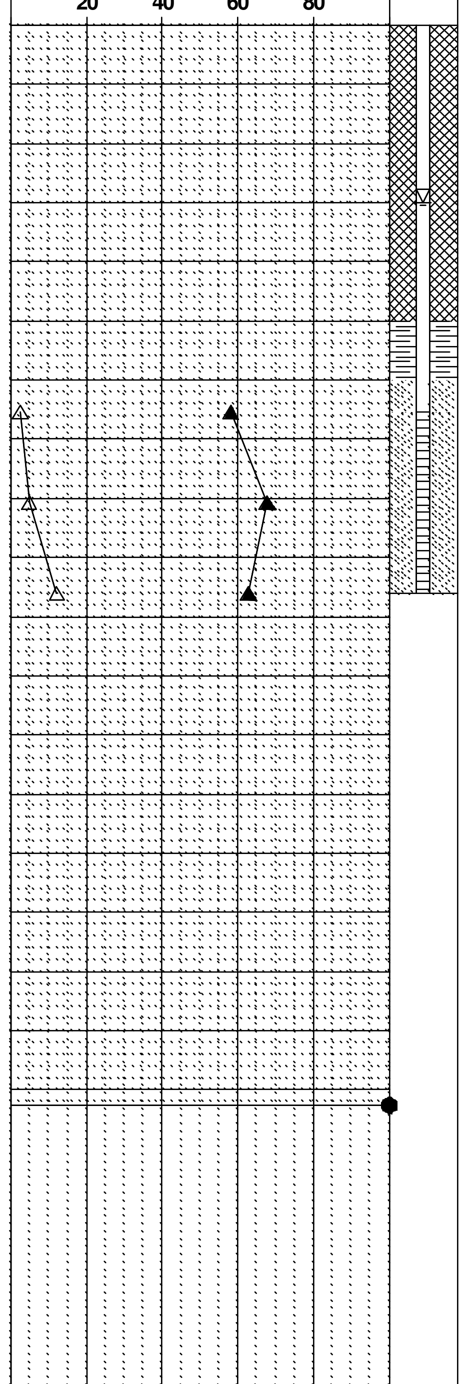
SOIL DESCRIPTION	Depth of boundary (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05				
FILL: Brown silty sand with gravel and bricks	1.22				
Stiff, grey **SILTY CLAY**	9.60	SS	1	83	5
		SS	2	75	3
		SS	3	100	2
		SS	4	100	P
Dynamic Cone Penetration Test commenced at 9.60m depth. Cone pushed to 18.3m depth.	18.26				
End of Borehole					
Practical DCPT refusal at 18.26m depth.					
(GWL @ 3.0m depth based on field observations)					

DEPTH (m)	ELEV. (m)
0	71.40
1	70.40
2	69.40
3	68.40
4	67.40
5	66.40
6	65.40
7	64.40
8	63.40
9	62.40
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	53.40

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

○ Water Content %

Piezometer Construction



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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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

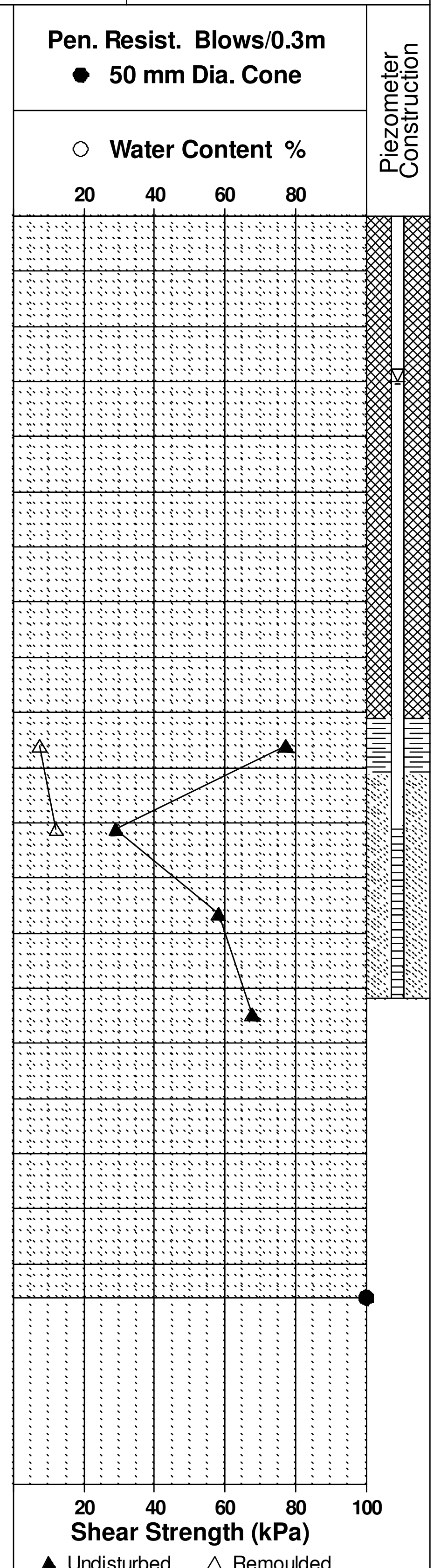
SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08				
FILL: Brown silty sand with gravel and bricks	1.37				
Stiff, grey **SILTY CLAY**		SS	1	33	9
		SS	2	83	4
		SS	3	100	1
		SS	4	100	2
		SS	5	100	3
		SS	6	75	P
- firm to stiff by 10.7m depth					
Stiff, grey **SILTY CLAY**	14.17				
Dynamic Cone Penetration Test commenced at 14.17m depth. Cone pushed to 18.6m depth.	19.61				

End of Borehole

Practical DCPT refusal at 19.61m depth.

(GWL @ 3.0m depth based on field observations)

DEPTH (m)	ELEV. (m)
0	71.85
1	70.85
2	69.85
3	68.85
4	67.85
5	66.85
6	65.85
7	64.85
8	63.85
9	62.85
10	61.85
11	60.85
12	59.85
13	58.85
14	57.85
15	56.85
16	55.85
17	54.85
18	53.85
19	52.85



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

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

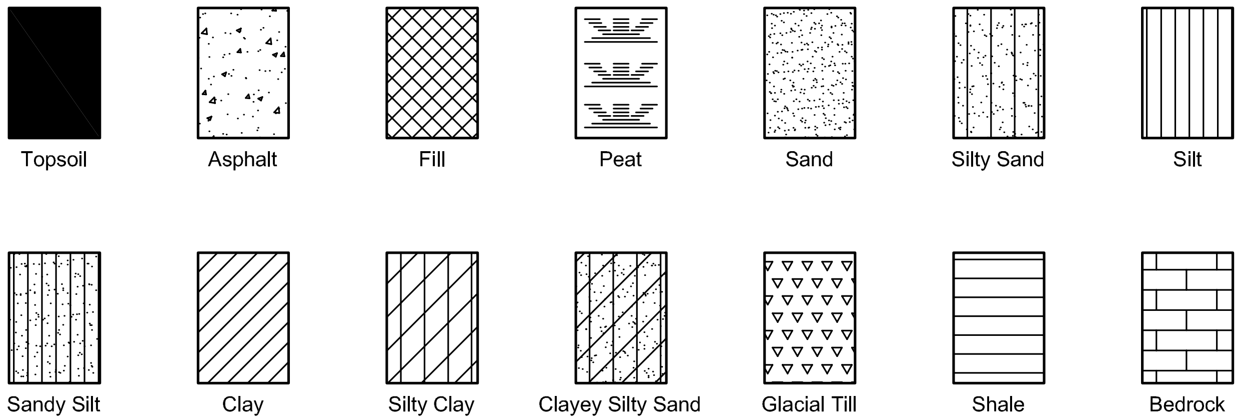
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

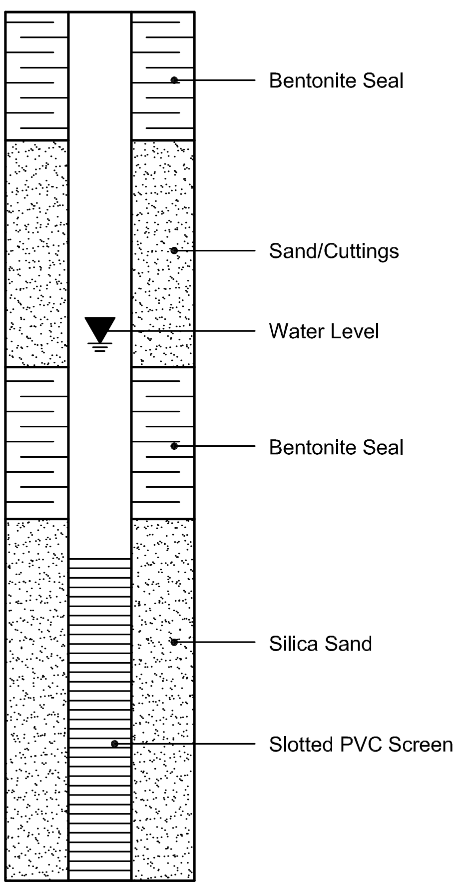
SYMBOLS AND TERMS (continued)

STRATA PLOT

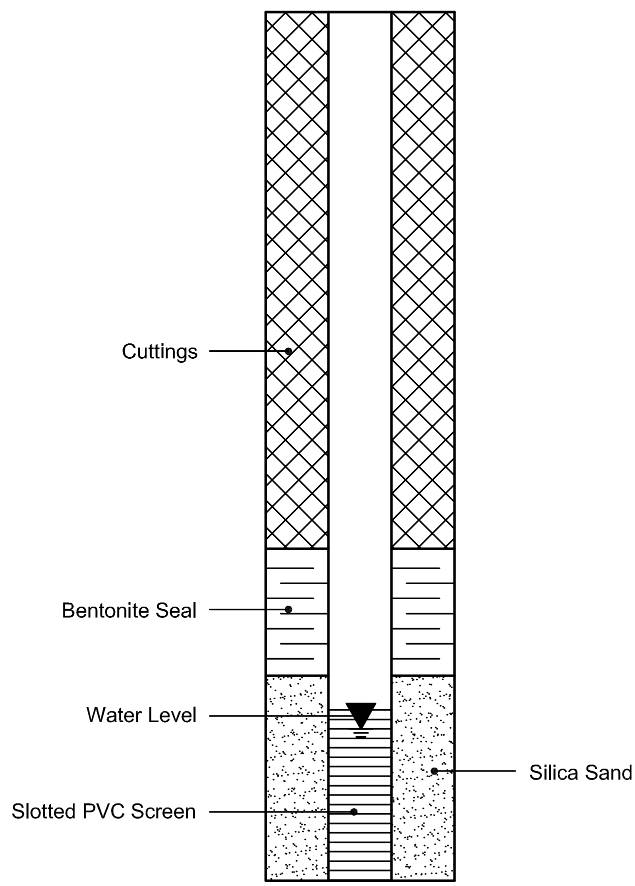


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - WATER SUPPRESSION SYSTEM

FIGURE 3 - PRESSURE RELIEF CHAMBER

FIGURE 4 - TEMPORARY DRAINAGE SYSTEM BELOW MUDSLAB

FIGURES 5 AND 6 - SHEAR WAVE VELOCITY TESTING PROFILES

DRAWING PG4985-1 - TEST HOLE LOCATION PLAN

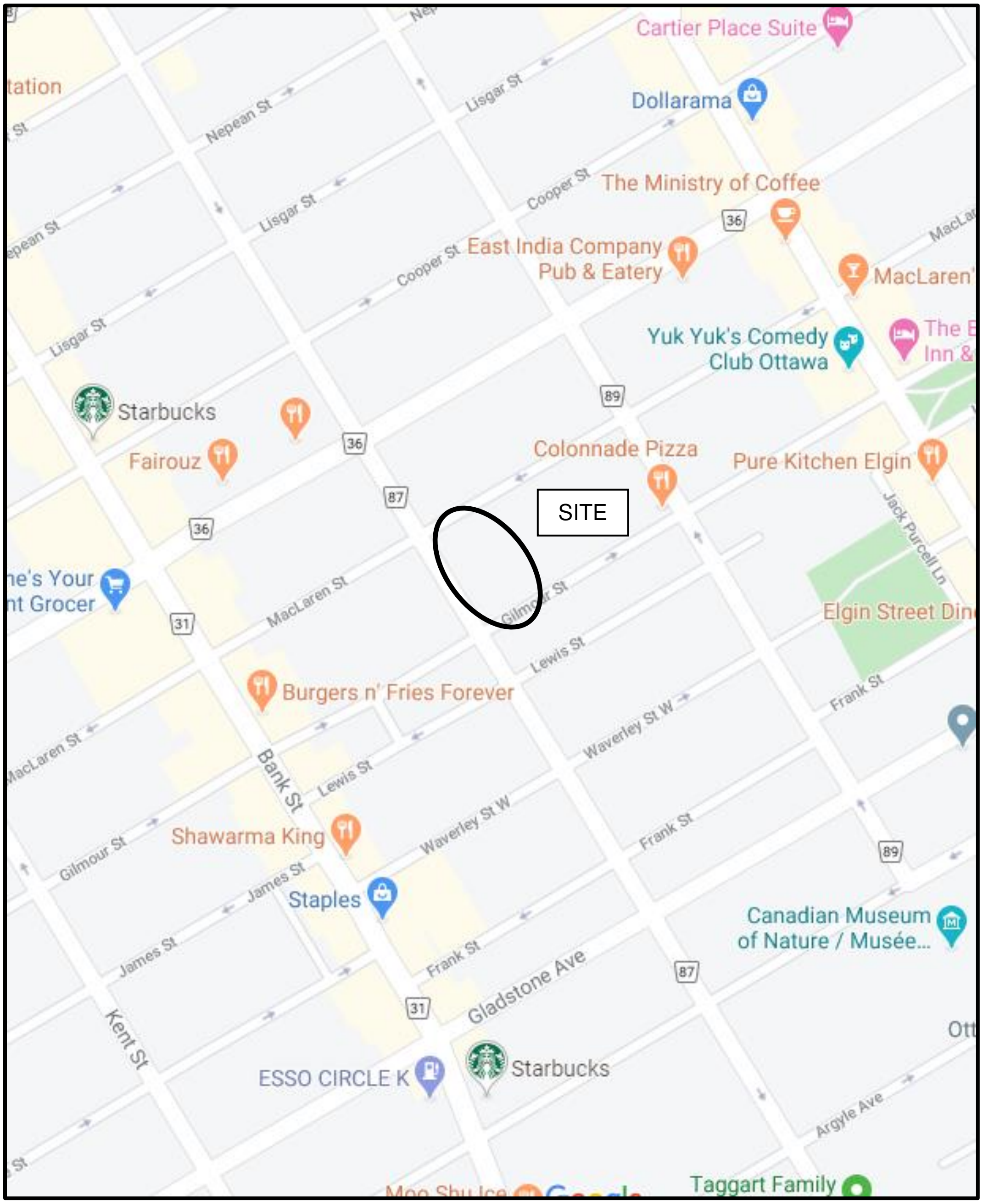


FIGURE 1

KEY PLAN

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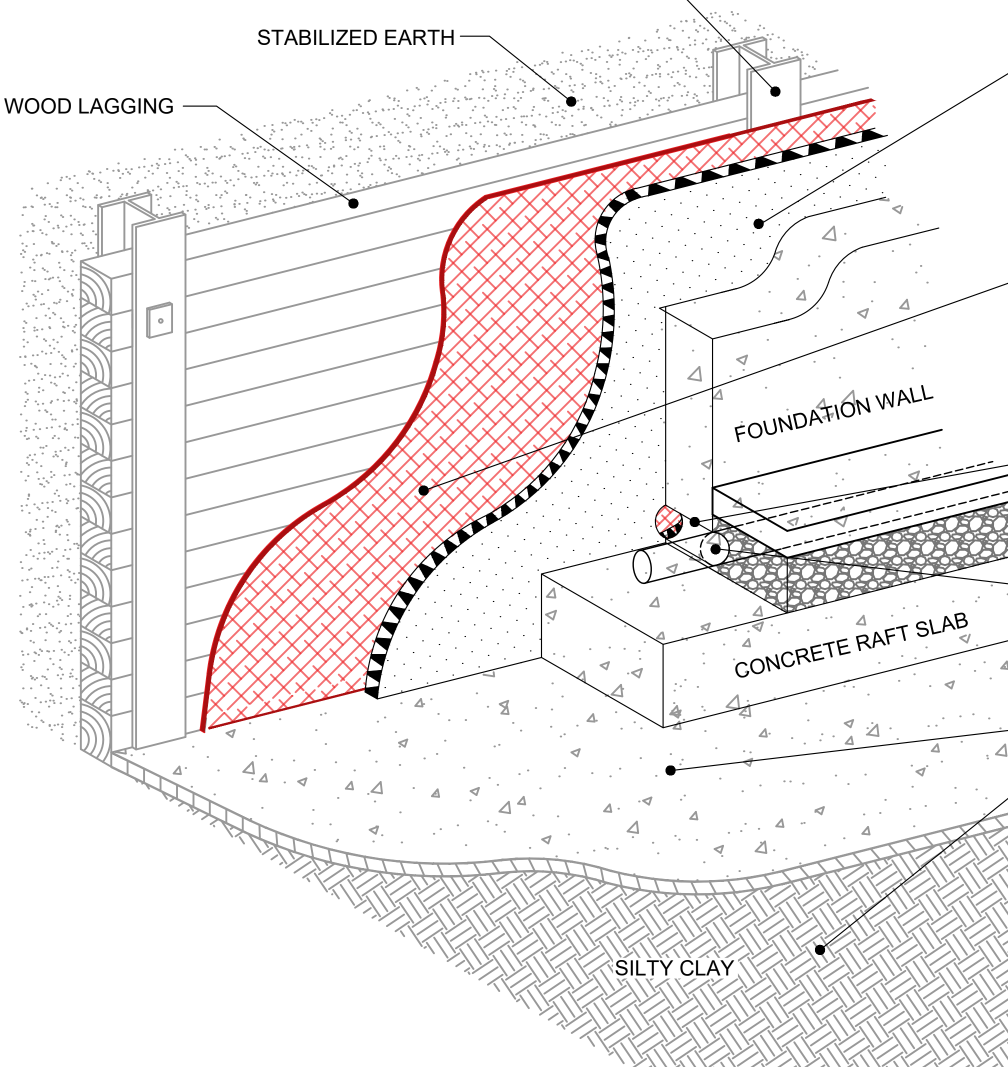

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TAGGART REALTY MANAGEMENT

PROPOSED HI-RISE BUILDINGS
267 O'CONNOR STREET

OTTAWA, ONTARIO

Title:

WATER SUPPRESSION SYSTEM

Scale: NTS	Date: 03/2020
Drawn by: RCG	Report No.: PG4985-1
Checked by: JV	Drawing No.: FIG.2
Approved by: FA	Revision No.:

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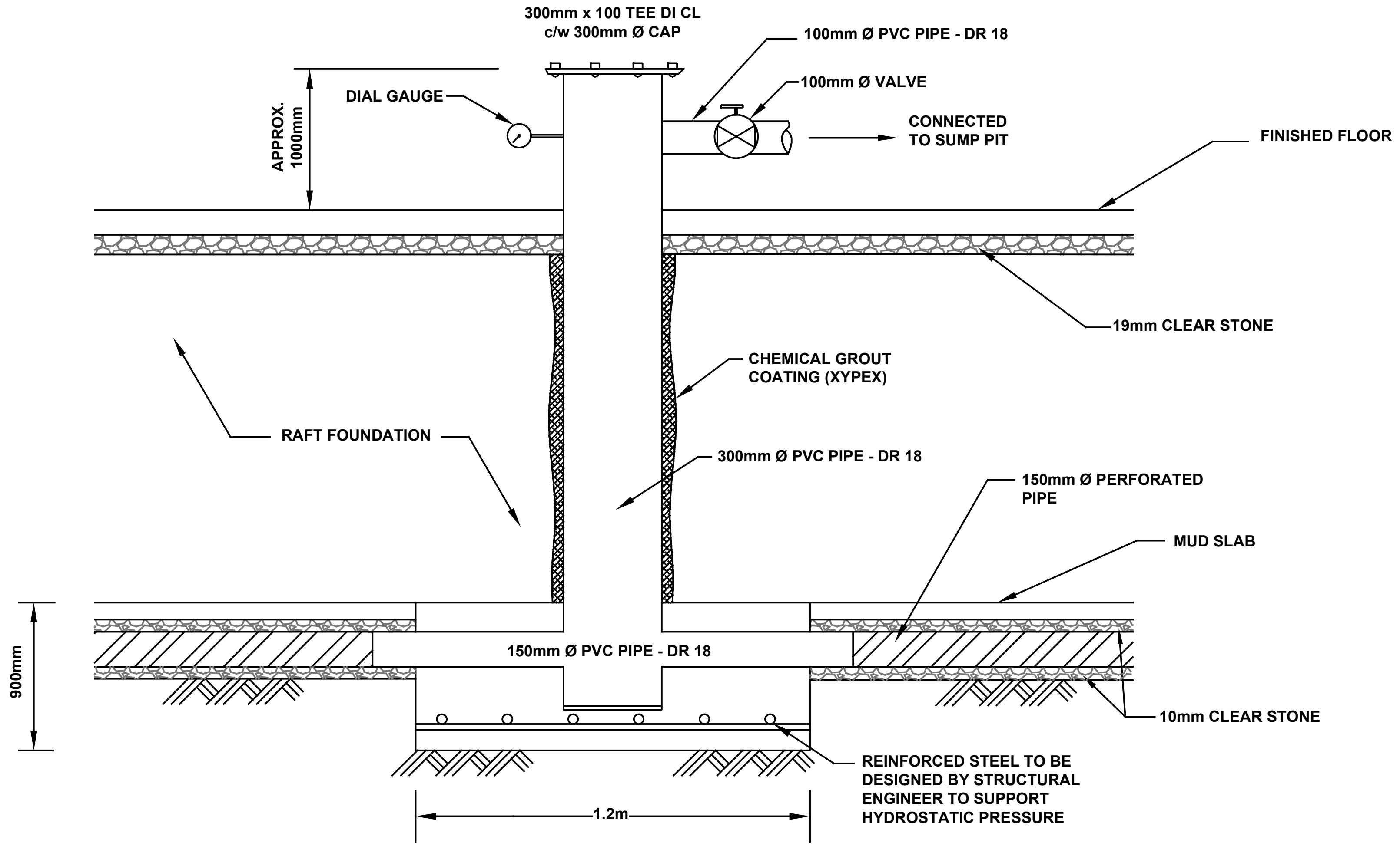

FIGURE 3
PRESSURE RELIEF CHAMBER

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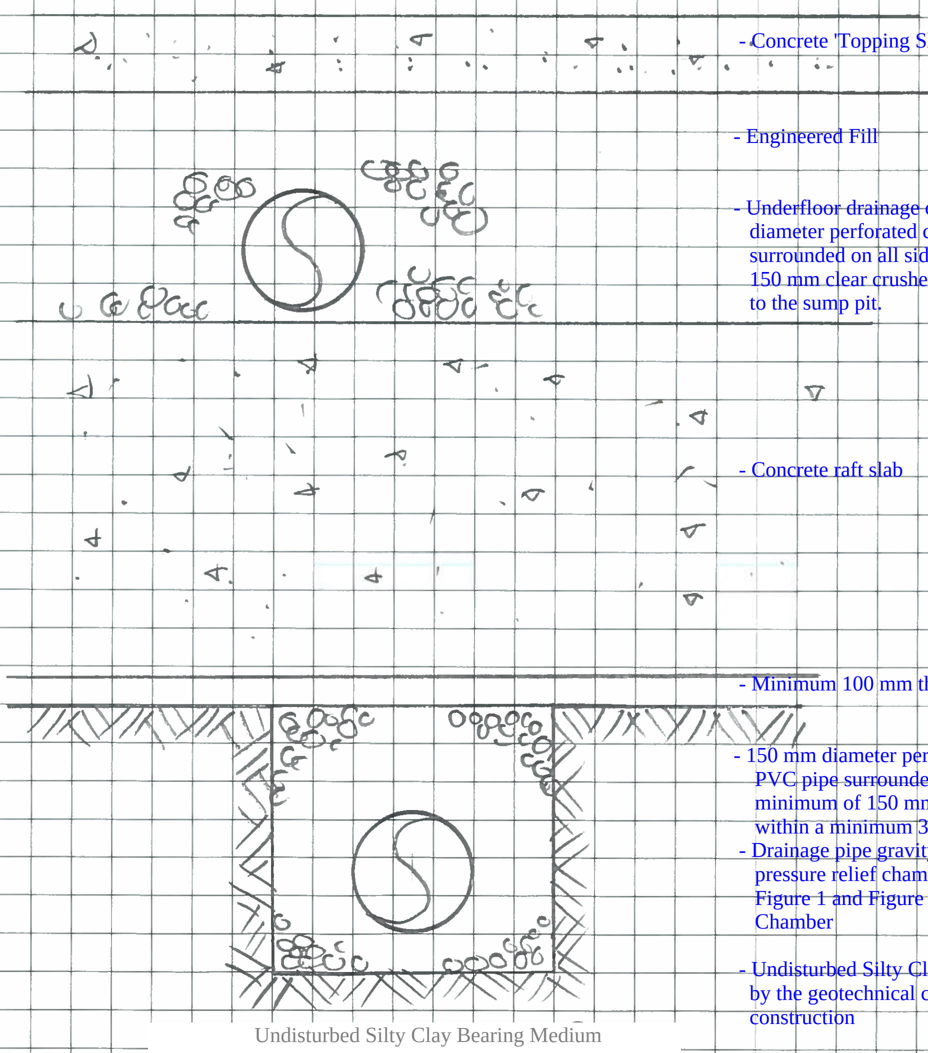
p:\geotechnical\pg49xx\pg4985 - taggart - 267 o'connor\pg4985 fig3 pressure relief chamber.dwg

patersongroup consulting engineers	Taggart Realty Management	Job No.	PG4985
	Proposed Hi-Rise Building	Date:	March 4, 2020
154 Colonnade Road South, Ottawa, ON, K2E 7J5	267 O'Connor Street	Dwn:	Figure 4
		Page:	NTS

Figure 4 - Temporary Drainage System Below Mudslab



- Concrete 'Topping Slab' (P4 Parking Level)
- Engineered Fill
- Underfloor drainage consisting of 150 mm diameter perforated corrugated PVC pipe surrounded on all sides by a minimum of 150 mm clear crushed stone gravity connected to the sump pit.
- Concrete raft slab
- Minimum 100 mm thick mud slab
- 150 mm diameter perforated corrugated PVC pipe surrounded on all sides by a minimum of 150 mm clear crushed stone within a minimum 300 mm deep trench.
- Drainage pipe gravity connected to the pressure relief chamber (refer to Figure 1 and Figure 2 - Pressure Relief Chamber
- Undisturbed Silty Clay Bearing Medium approved by the geotechnical consultant at the time of construction

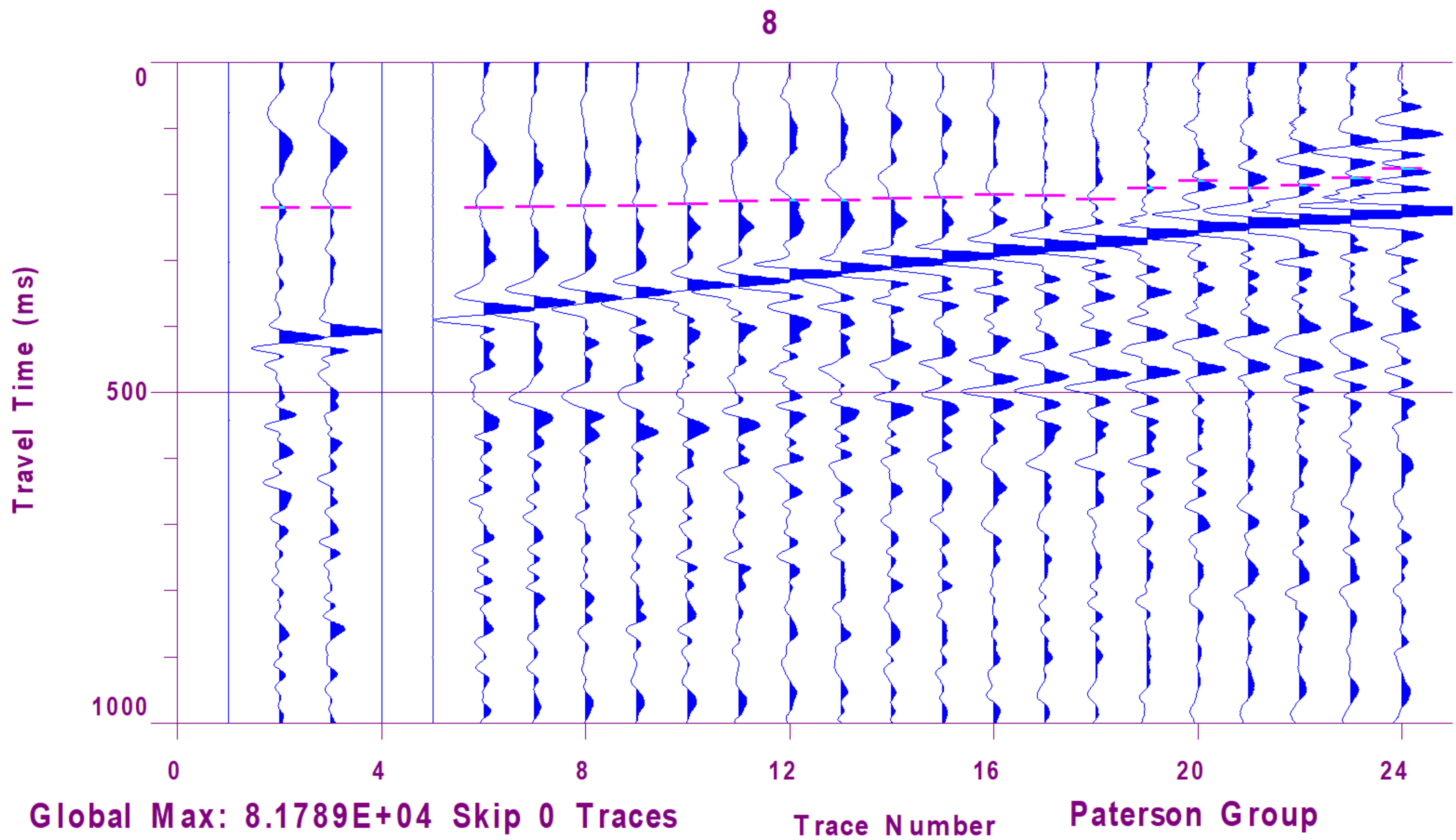


Figure 5 – Shear Wave Velocity Profile at Shot Location +69.5 m

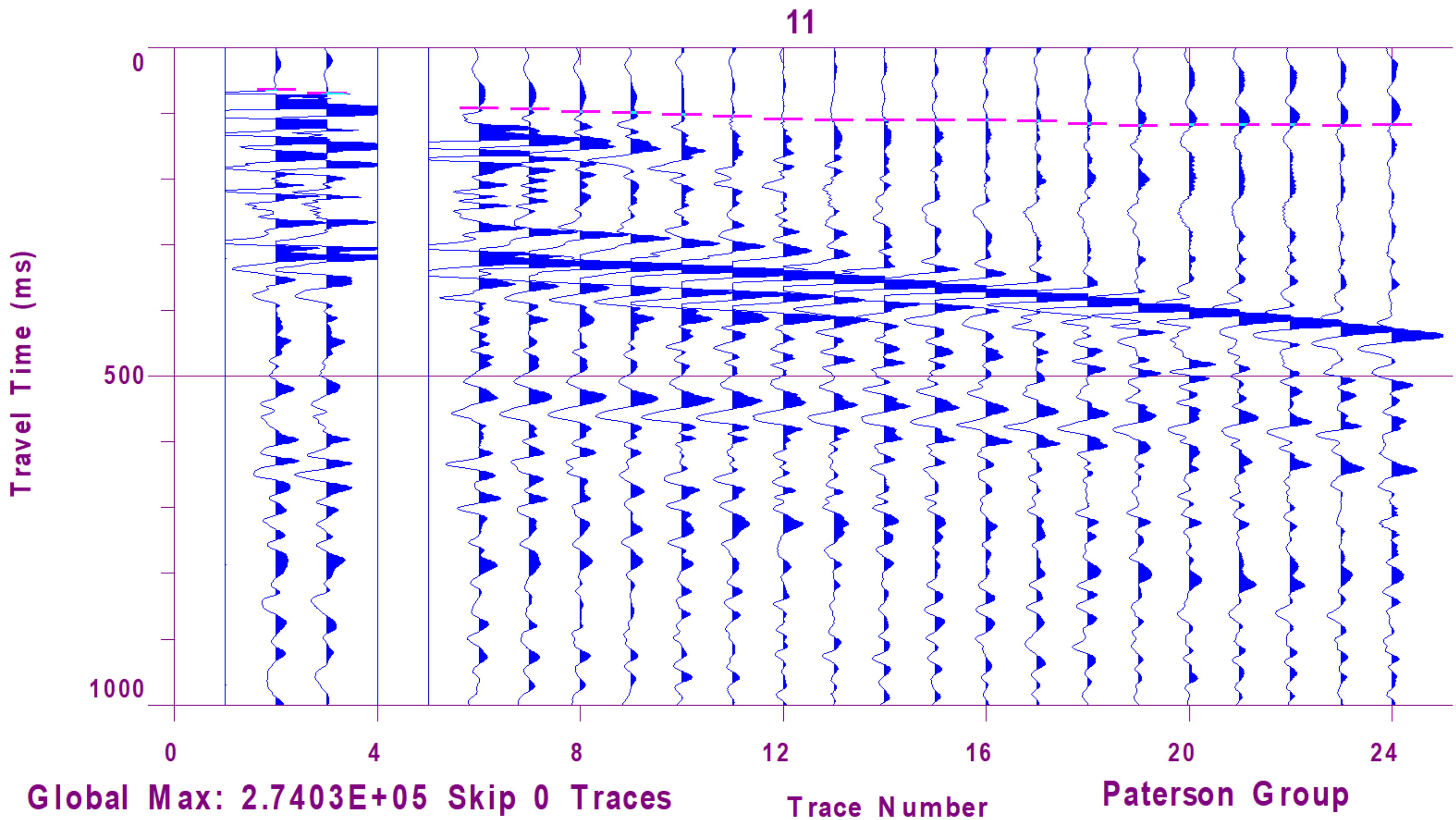
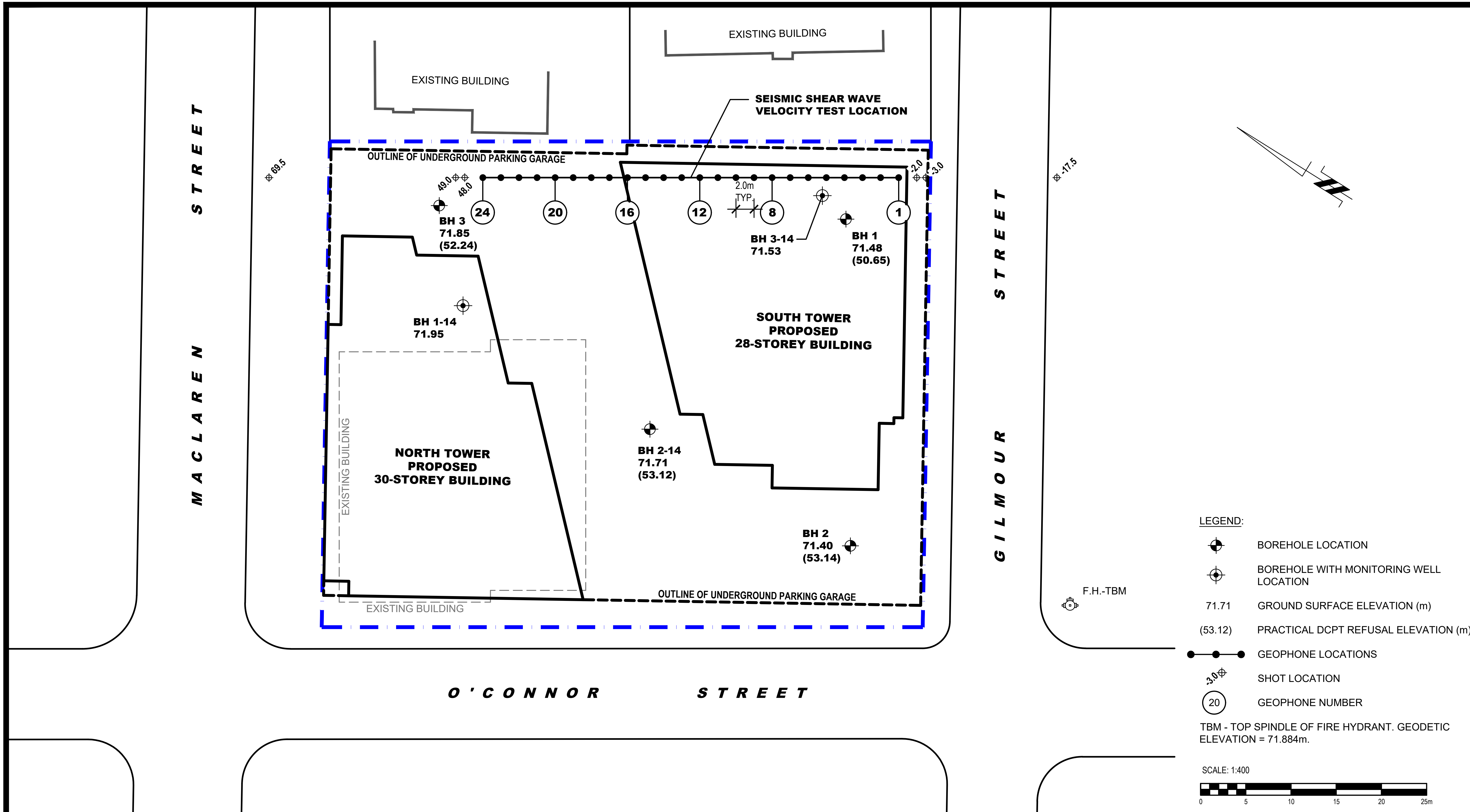


Figure 6 – Shear Wave Velocity Profile at Shot Location -2 m



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NO.	REVISIONS	DATE	INITIAL
1	SITE PLAN REVISED AND SEISMIC SURVEY INFORMATION ADDED	09/24/2020	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: JV	Dwg. No.: **PG4985-1**
Approved by: DJG	Revision No.: 1

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