Geotechnical Engineering

Environmental Engineering

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Geotechnical Investigation

Proposed Residential Development
335 Roosevelt Avenue
Ottawa, Ontario

Prepared For

Uniform Urban Developments

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1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Uniform Urban Developments (Uniform) to prepare a geotechnical report for a proposed residential development to be located at 335 Roosevelt Avenue 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 geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation, therefore, the present report does not address environmental issues. A Phase I-II was completed for this subject site by Paterson but is presented under a separate cover.

2.0 PROPOSED DEVELOPMENT

It is our understanding that the proposed residential development will consist of two and four high-rise and low-rise residential buildings, respectively. The two high-rise buildings are understood to be 18 and 21 storeys high, whereas the low-rise buildings will be 4 storeys high. It is further understood that the proposed basement levels will consist of two levels of underground parking which will extend to the property lines of the subject site.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on November 9 and 10, 2010. At that time, five (5) boreholes were advanced to a maximum depth of 9.5 m. The borehole locations were distributed in a manner to provide general coverage of the subject site. The locations of the boreholes are shown on Drawing PG2178-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down 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. In addition, bedrock was cored at each borehole location using diamond drilling procedures.

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.

Diamond drilling was carried out in each borehole to determine the nature of the bedrock. Total core recovery (TCR) and rock quality designation (RQD) values were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The TCR value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the core run. The RQD value is the ratio, in percentage, of the total length of rock pieces longer than 100 mm in one core run over the length of the core run. Each of these values are indicative of the quality of the bedrock.

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

Groundwater

A flexible polyethylene standpipe was installed in BH 1, BH 2 and BH 4. PVC monitoring wells (50 mm diameter) were installed in BH 3 and BH 5. These were installed to permit the monitoring of the groundwater level subsequent to the completion of the sampling program.

3.2 Field Survey

The borehole locations were selected, determined in the field and surveyed by Paterson. The ground surface elevation at each borehole location was referenced to a temporary benchmark (TBM), consisting of a magnetic nail in a utility pole. A geodetic elevation of 67.30 m has been provided to the TBM by Annis O'Sullivan Vollebekk Ltd. The location of the TBM and boreholes, as well as, the ground surface elevation at each borehole are presented on Drawing PG2178-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

At the time of the field program, three (3) existing buildings were present on the subject site. The remainder of the site was asphalt covered with the exception of a gravel area on the south portion of the property.

The site is bordered to the north by the transitway, to the west by Roosevelt Avenue, to the south by Winston Avenue and Wilmont Avenue, and to the east by a 7 storey residential building. The westernmost building was noted to be approximately 0.6 m below Roosevelt Avenue. Additionally, the transit-way located north of the subject site was noted to be approximately 6 m below the elevation of 335 Roosevelt Avenue. The subject site is relatively flat.

4.2 Subsurface Profile

The subsurface profile at the borehole locations consist of either asphaltic concrete or silty sand fill overlying fill consisting of silty sand with some gravel and clay. Native silty clay or silt was encountered below the fill material at most of the boreholes. Bedrock was encountered at depths between 0.7 and 1 m depths. Specific details of the soil profile at each borehole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

The bedrock was cored at all borehole locations to determine its nature and quality. Based on the results of coring, the bedrock consists of limestone with layers of black shale. Values for TCR and RQD were calculated for each rock core and the quality of the bedrock was assessed based on these results.

Based on the observations, the upper 0.5 to 2 m of the bedrock is of poor to fair quality while the lower portion of the core is of good to excellent quality. The bedrock consists of limestone with interbedded shale, with a black shale limestone extending through the rock at depths between 1.5 and 3 m.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and dolomite of the Gull River formation, which is encountered at depths varying between 1 and 2 m.

4.3 Groundwater

Groundwater levels (GWL) were measured in all boreholes on November 16, 2010. The measured GWL readings are presented in Table 2. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels		Recording Date
		Depth (m)	Elevation (m)	
BH 1	66.39	4.88	61.51	November 16, 2010
BH 2	66.37	6.53	59.84	November 16, 2010
BH 3	66.43	Dry	--	November 16, 2010
BH 4	66.64	3.84	62.80	November 16, 2010
BH 5	66.50	4.97	61.53	November 16, 2010

5.0 DISCUSSION

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed multi-storey buildings. The proposed buildings are expected to be founded on conventional spread footings placed on clean, surface sounded bedrock.

Considering that the site is underlain by shallow bedrock (within 1 m of the surface), shoring may not be necessary if the excavation of the overburden soils can be stepped back from the bedrock excavation face. Temporary rock bolts may be required to stabilize the walls of the excavation through bedrock.

Bedrock removal will be required to complete the two (2) levels of underground parking. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

A watermain alignment runs along the north property boundary in close proximity of the subject site. It is expected that the adjacent watermain could be subjected to potential vibrations associated with the bedrock blasting program. To ensure that no detrimental vibrations cause damage to the adjacent watermain, a vibration monitoring and control program is recommended to be undertaken during the blasting and excavation work required for the proposed building excavation.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the relatively shallow bedrock depth at the subject site and the anticipated founding level for the proposed building, all existing overburden material will be excavated from within the proposed building footprint. Bedrock removal will be required for the construction of the parking garage levels.

Bedrock Removal

Based on the bedrock encountered in the area, it is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm per second during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock could be completed with almost vertical side walls. Where bedrock is of lower quality, the excavation face should be free of any loose rock. An area specific review should be completed by the geotechnical consultant at the time of construction to determine if rock bolting or other remedial measures are required to provide a safe excavation face for areas where low quality bedrock is encountered.

An existing watermain alignment is located approximately 2.5 m north of the subject site's north property line. Blasting can be used for most of the bedrock removal up to a minimum horizontal distance of 2 m from the outer edge of the existing watermain. It is recommended that bedrock removal be completed by hoe ramming and grinding techniques within 2 m from the watermain. Blasting operations will be reviewed and the 2 m minimum distance from the watermain may be increased if vibrations from the blasting operation are questionable.

Vibration monitors should be installed to measure the vibrations and to ensure that the vibration levels stay below 25 and 15 mm/s at the property boundary and watermain, respectively.

Vibration Considerations

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

The following construction equipments could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system with soldier piles or sheet piling will require these pieces of equipments. Vibrations, caused by blasting or construction operations could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the recommended vibration limit, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Vibration Monitoring and Control Plan

To ensure that no disturbance to the existing watermain occurs, a vibration monitoring and control plan (VMCP) is recommended during the excavation program. The purpose of the vibration monitoring and control plan is to provide measures to be implemented by the contractor to manage excavation operations and any other vibration sources during the construction for the proposed development. The VMCP will also provide a guideline for assessing results against the relevant vibration impact assessment criteria and recommendations to meet the required limits.

The monitoring program will incorporate real time results at the existing watermain segment adjacent to the subject site. The monitoring equipment should consist of a tri-axial seismograph, capable of measuring vibration intensities up to 254 mm/s at a frequency response of 2 to 250 Hz. At least two vibration monitoring devices should be placed adjacent to the existing watermain. It is recommended that the vibration monitoring devices be installed at invert level of the existing watermain and periodically inspected during the construction program.

A copy of the geotechnical report, which includes the VMCP should be provided to all parties involved with the construction for review. A meeting between Paterson and site contractor should be conducted prior to any excavation or construction of the subject site to review the following:

❏ Review the pre-condition/pre-construction survey;
❏ Control measures (i.e vibrations, noise);
❏ Monitoring locations;
❏ Tracking and reporting of excavation progress, and;
❏ Review procedure for exceedances (i.e vibrations, noise), complaints, evaluation and corrective measures.

When an event is triggered, Paterson will review the results and provide any necessary feedback. Otherwise, the vibration results will be summarized in the weekly report. The following table outlines the vibration limits for the adjacent watermain segment.

Table 2 - Structure Vibration Limits for adjacent Watermain Segment

Dominant Frequency Range (Hz)	Peak Particle Velocity (mm/s)	Event	Description of Event
<10	all	none	no action required
<40	>10	trigger level	Warning e-mail sent to contractor.
<40	≥15	exceedance level	Exceedance e-mail and phone call to the contractor. All operations are ceased to review on-site activities.
>40	>15	trigger level	Warning e-mail sent to contractor.
>40	≥20	exceedance level	Exceedance e-mail and phone call to the contractor. All operations are ceased to review on-site activities.

The monitoring protocol should include the following information:

Trigger Level Event

- ❑ Paterson will review all vibrations over the established warning level, and;
- ❑ Paterson will notify the contractor if any vibration occur due to construction activities and are close to exceedance level.

Exceedance Level Event

- ❑ Paterson will notify all the relevant stakeholders via email;
- ❑ Ensure monitors are functioning, and;
- ❑ Issue the vibration exceedance result.

Fill Placement

It is expected that a concrete slab will be poured directly over bedrock; therefore, fill used for grading beneath building will not be required, other than around the footings, as required.

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. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective standard Proctor maximum dry density (SPMDD).

Excavated shale deteriorates upon exposure to air and is not generally suitable for re-use as an engineered fill.

5.3 Foundation Design

It is understood that footings will be founded on bedrock. Footings placed on a clean, surface sounded bedrock surface at this elevation can be designed using a bearing resistance value at serviceability limit states (SLS) of **1,000 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

A bearing resistance value at SLS of **2,000 kPa** and a factored bearing resistance value at ULS of **3,500 kPa** could be used if the bedrock is free of seams, fractures and voids within 1.5 m below the bedrock surface. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level along the footing alignments. The drill holes should be spaced on about a 10 m grid interval or one (1) hole per significant pad footing. The drill hole inspection should be carried out by the geotechnical consultant.

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 sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Settlement

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

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 building from Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are attached to the present letter.

Field Program

The shear wave testing location is presented in Drawing PG2178-1 - Seismic Array Location Plan attached to this report. Paterson field personnel placed 24 horizontal geophones in a straight line in roughly a north-south orientation. The 4.5 Hz. horizontal geophones were mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 1 m intervals and 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 1, 2 and 10.5 m away from the first and last geophone.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. 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 building's foundation. 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 velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

It is understood that the footings of the proposed building are to be founded directly on the bedrock surface. Based on our analysis, the bedrock shear wave was calculated to be 2,220 m/s.

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012, as presented below.

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

$$V_{s30} = \frac{30m}{\left(\frac{30m}{2{,}220m/s}\right)}$$

$$V_{s30} = 2{,}220m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for shallow foundations located at the subject site is 2,220 m/s. Therefore, a **Site Class A** is applicable for the proposed building, as per Table 4.1.8.4.A of the OBC 2012.

5.5 Basement Wall

It is understood that the basement walls are to be poured against a waterproofing system, which will be placed against the exposed bedrock face. A nominal coefficient of at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

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 20 kN/m^3. The applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil, 0.5
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.6 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 of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

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.

It should be further noted that centre to centre spacing between bond lengths be at least four (4) times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service. To resist seismic uplift pressures, a passive rock anchor system can be used. It should be noted that a post-tensioned anchor will take the uplift load with much less deflection than a passive anchor.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall 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 plastic sleeve to act as a bond break.

Grout to Rock Bond

Generally, the unconfined compressive strength of limestone ranges between 60 and 120 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. Based on existing subsoils information, a **Rock Mass Rating (RMR) of 69** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

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 3 - 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) - Good quality Limestone Hoek and Brown parameters	69 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	60 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
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 4.

Table 4 - 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	1.2	0.6	1.8	250
	1.9	0.8	2.7	500
	3	1.5	4.5	1000
125	1.1	0.5	1.6	250
	1.5	0.7	2.2	500
	2.6	1	3.6	1000

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

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.

5.7 Pavement Design

Asphalt pavement is not anticipated to be required at the subject site. However, should pavement be reconsidered for the project, the recommended pavement structures shown in Tables 5 and 6 would be applicable.

Table 5 - Recommended Pavement Structure - Car Only Parking Areas	
Thickness mm	**Material Description**
50	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill

Table 6 - Recommended Pavement Structure - Access Lanes	
Thickness mm	**Material Description**
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - Either fill, in situ soil 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 Ontario Provincial Standard Specification (OPSS) Granular B Type I or Type II material.

The pavement granulars (base and subbase) should be placed in maximum 300 mm thick layers and compacted to a minimum of 100% of the materials' SPMDDs using suitable compaction equipment.

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. It is understood that insufficient room is available for exterior backfill below the bedrock surface. The following system is suggested:

❑ Bedrock vertical surface
❑ Metal “V” pan
❑ Composite drainage layer

It is recommended that the composite drainage system (such as Miradrain G100N or equivalent) extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor drainage may be required to control water infiltration due to groundwater lowering within the bedrock. For design purposes, we recommend that 100 or 150 mm in perforated pipes be placed at 3 to 4.5 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter footings 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 in this regard.

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.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should 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.

6.4 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. A minimum of 300 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on bedrock subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 300 mm thick lifts compacted to a minimum of 99% of the material’s SPMDD.

It is expected that the silt may be used above cover material if the excavation operations are carried out in dry weather conditions. Well fractured bedrock should be acceptable as backfill provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones 200 mm or larger in their longest dimension are removed.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD. No stones 200 mm or greater in their longest dimension should be reused. Within the frost zone (1.8 m below finished grade), non frost susceptible materials should be used when backfilling trenches below the original bedrock level.

6.5 Groundwater Control

Groundwater Control for Building Construction

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.

Infiltration levels are anticipated to be low through the excavation face. The groundwater infiltration will be controllable with open sumps and pumps.

A temporary MECP permit to take water (PTTW) will be required for this project if more than 50,000 L/day are to be pumped during the construction phase. A minimum of four to five months should be allocated for completion of the application and issuance of the permit by the MECP.

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 cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (i.e.- less than 50,000 L/day) with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

6.6 Winter Construction

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

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and/or the footings are protected with sufficient soil cover to prevent freezing at the founding level. Placing concrete directly over cold bedrock surfaces is not recommended.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice in the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. Also, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure.

Precaution should be taken where excavations are carried out in close 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 documents to protect the walls of the excavations from freezing, if applicable.

6.7 Protection of Existing Watermain

During the bedrock removal program for the proposed development, the existing watermain located just beyond the north property boundary of the subject site will require protection.

Bedrock Condition

Based on our existing information, the bedrock is expected at approximate elevation +/- 65.5 m. The upper portion of the bedrock is weathered and the bedrock quality improves with depth. The bedrock quality is generally fair to good based on the rock quality designation (RQD) findings below upper 1 to 2 m of weathered bedrock.

Paterson undertook a test pit excavation program on the subject property along the northern boundary on September 13, 2010. Three test pits were excavated using a rubber tired backhoe and our findings can be summarized as follows:

Subsurface Conditions	Test Pit 1	Test Pit 2	Test Pit 3
Pavement structure overlying sandy silt deposit thickness	810 mm	810 mm	710 mm
Weathered bedrock thickness	100 mm	none	none
Sound bedrock depth	910 mm	810 mm	710 mm

The approximate locations of the test pits are shown on Drawing PG2178-1 - Test Hole Location Plan in Appendix 2.

Bedrock Removal along the Northern Boundary

The bedrock removal for the subject site will be carried out using a combination of blasting and hoe-ramming techniques, especially along the northern boundary where the existing watermain is located. The bedrock removal along the northern boundary will be carried out as follows:

- Blasting can be used for most of the bedrock removal up to a minimum horizontal distance of 2 m from the outer edge of the existing watermain. A minimum line drilling spacing of 300 mm c/c will be required at the 2 m blasting boundary limit.
- The blasting contractor will control the blasting operation to keep peak particle velocities below 25 mm/s at the property boundary. It is expected that the blasting contractor will commence the blasting operation at the opposite end of the site so that blasting patterns and vibrations can be monitored and verified prior to attempting any blasting along the northern boundary adjacent to the existing watermain. This approach will allow the blasting contractor to adjust and control the blasting operation.
- Blasting operations will be reviewed and the 2 m minimum distance from the watermain may be increased if vibrations from the blasting operation are questionable.

- Within the minimum 2 m distance from the watermain, the bedrock will be removed using hoe-ramming or grinding techniques. Blasting will not be permitted. Line drilling spacing will be decreased to 200 mm c/c along the proposed excavation boundary. Similar to the blasting operations, hoe-ramming or grinding operations will be governed by the vibrations they produce along the property boundary adjacent to the watermain.

Monitoring and Reporting

- Two seismographs will be installed directly on the bedrock along the northern property line to monitor vibrations. Each blasting event will be reviewed and reported to the blasting contractor and the site superintendent.

- A weekly summary report will be issued presenting our findings and observations. Any concerns identified during the monitoring will be immediately reported, as discussed in Subsection 5.2, and the rock removal operations in the immediate area will be temporarily halted to address the concern.

7.0 RECOMMENDATIONS

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

- ❑ Review of the bedrock excavation faces and the installation of the rock anchors, if applicable.
- ❑ 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 3.0 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Density tests to determine the level of compaction achieved.

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

8.0 STATEMENT OF LIMITATIONS

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review our recommendations when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and all borehole logs are furnished as a matter of general information only and borehole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Uniform Urban Developments and 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
July 26, 2020
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- ❏ Uniform Urban Developments (3 copies)
- ❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 9 November 2010

FILE NO. PG2178

HOLE NO. BH 1

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.39
Asphaltic concrete	0.05	AU	1				
FILL: Dark brown silty sand, some gravel	0.46						
Grey-brown SILTY CLAY, some sand	0.74	SS	2		50+		
BEDROCK: Grey limestone - black shaley limestone from 1.5 to 2.5m depth		RC	1	94	22	1	65.39
		RC	2	100	51	2	64.39
						3	63.39
		RC	3	100	78	4	62.39
		RC	4	100	84	5	61.39
						6	60.39
		RC	5	100	35	7	59.39
		RC	6	100	96	8	58.39
						9	57.39
	9.47	RC	7	100	100		
End of Borehole (GWL @ 4.88m-Nov. 16/10)							

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

○ Water Content %

20 40 60 80

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

Piezometer Construction

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

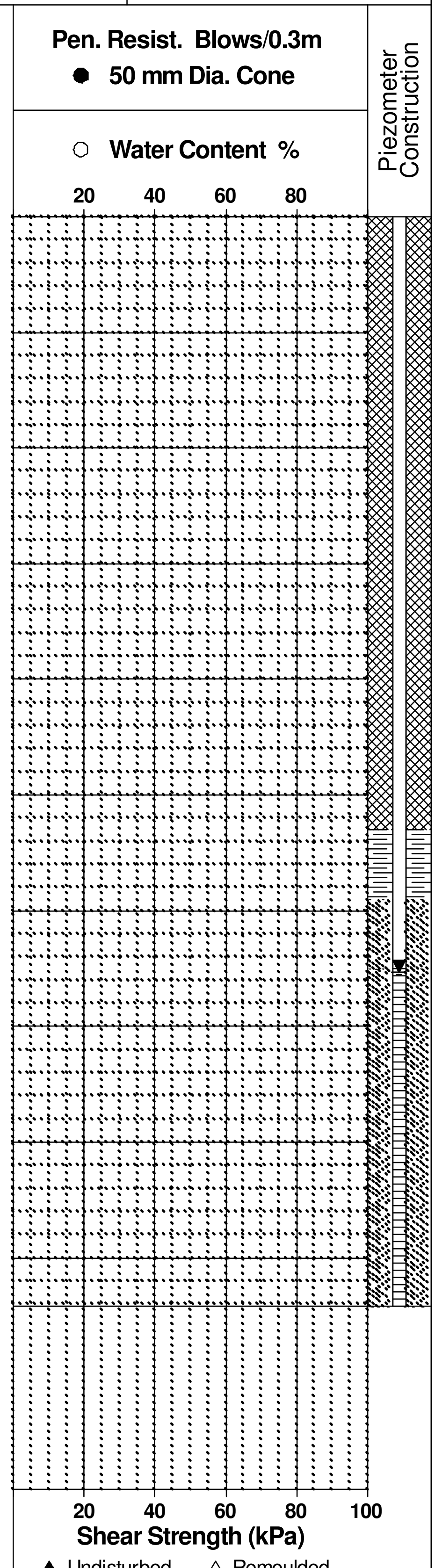
BORINGS BY CME 55 Power Auger

DATE 9 November 2010

FILE NO. PG2178

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.37
Asphaltic concrete	0.05						
FILL: Dark brown silty sand,	0.46	AU	1				
some gravel	0.60						
Grey-brown **SILTY CLAY**		SS	2	0	50+		
		RC	1	84	58	1	65.37
BEDROCK: Grey limestone		RC	2	100	57	2	64.37
- black shaley limestone from 1.6m to 2.5m depth						3	63.37
		RC	3	98	83	4	62.37
		RC	4	100	92	5	61.37
						6	60.37
		RC	5	98	98	7	59.37
		RC	6	100	93	8	58.37
						9	57.37
	9.42	SS	7	100	100		
End of Borehole							
(GWL @ 6.53m-Nov. 16/10)							



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 9 November 2010

FILE NO. PG2178

HOLE NO. BH 3

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.43
Asphaltic concrete	0.05						
FILL; Brown silty sand with gravel	0.60	AU	1				
FILL: Light brown silty sand, some gravel, trace clay	0.97	SS	2	0	50+		
BEDROCK: Grey limestone - black shaley limestone from 1.5m to 1.7m depth		RC	1	88	63	1	65.43
		RC	2	97	35	2	64.43
						3	63.43
		RC	3	100	75	4	62.43
		RC	4	98	87	5	61.43
						6	60.43
		RC	5	98	85	7	59.43
		RC	6	100	89	8	58.43
	9.40	RC	7	96	96	9	57.43
End of Borehole (BH dry - Nov. 16/10)							

STRATA PLOT

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

○ Water Content %

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

Piezometer Construction

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 10 November 2010

FILE NO. PG2178

HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.64
Asphaltic concrete	0.04						
Crushed stone	0.15	AU	1				
FILL: Brown silty sand, some gravel	0.91						
Grey-brown **SILT**	1.17	SS	2	90	10	1	65.64
BEDROCK: Shaley limestone	3.00	RC	1	92	36	2	64.64
						3	63.64
BEDROCK: Grey limestone	9.37	RC	2	100	55	4	62.64
		RC	3	100	60	5	61.64
						6	60.64
		RC	4	100	93	7	59.64
		RC	5	100	92	8	58.64
		RC	6	100	100	9	57.64
End of Borehole							
(GWL @ 3.84m-Nov. 16/10)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 10 November 2010

FILE NO. PG2178

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.50
FILL: Dark brown silty sand with gravel, trace metals, brick and asphalt — 0.76		SS	1	50	8		
FILL: Black silty sand — 0.94		SS	2	78	50+		
Light brown **SILT** — 0.99		RC	1	86	0	1	65.50
BEDROCK: Grey limestone - shaley limestone from 1.9m to 2.0m depth		RC	2	97	45	2	64.50
						3	63.50
		RC	3	98	85		
						4	62.50
		RC	4	100	77	5	61.50
						6	60.50
		RC	5	100	95		
						7	59.50
		RC	6	100	82	8	58.50
		RC	7	100	83	9	57.50
9.40 End of Borehole							
(GWL @ 4.97m-Nov. 16/10)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

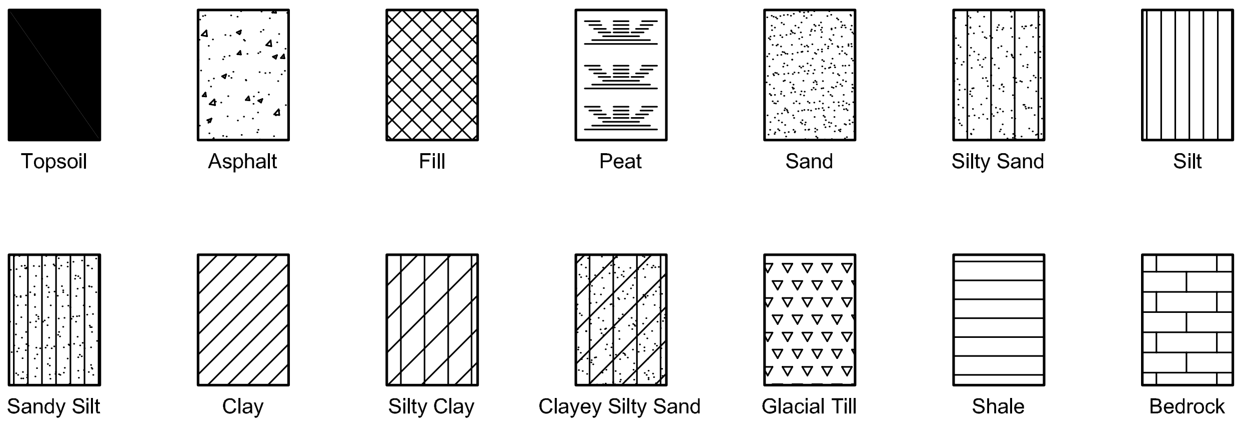
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

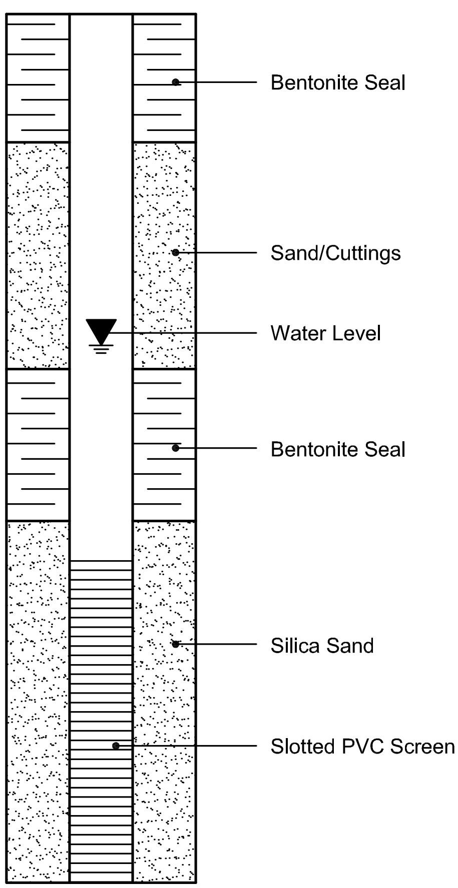
SYMBOLS AND TERMS (continued)

STRATA PLOT

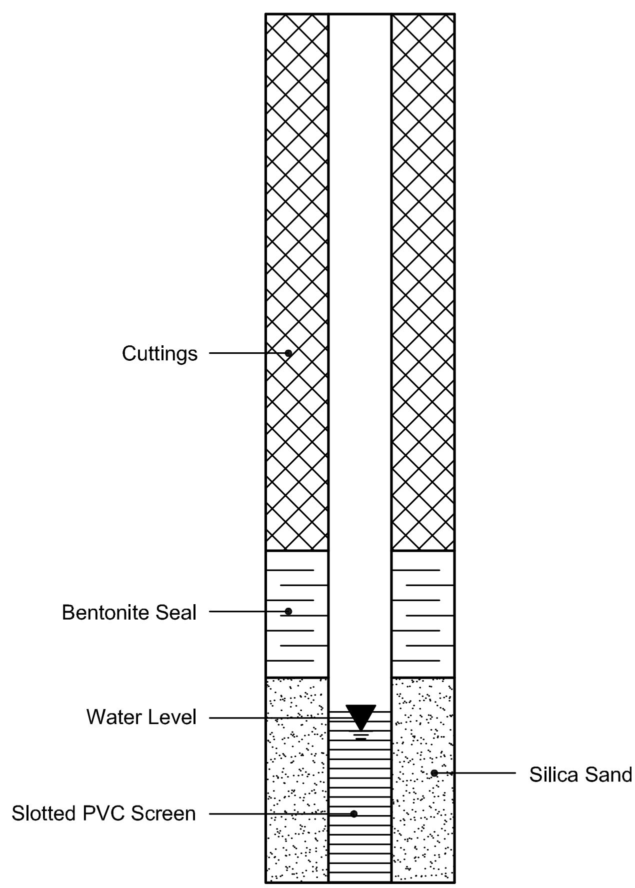


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



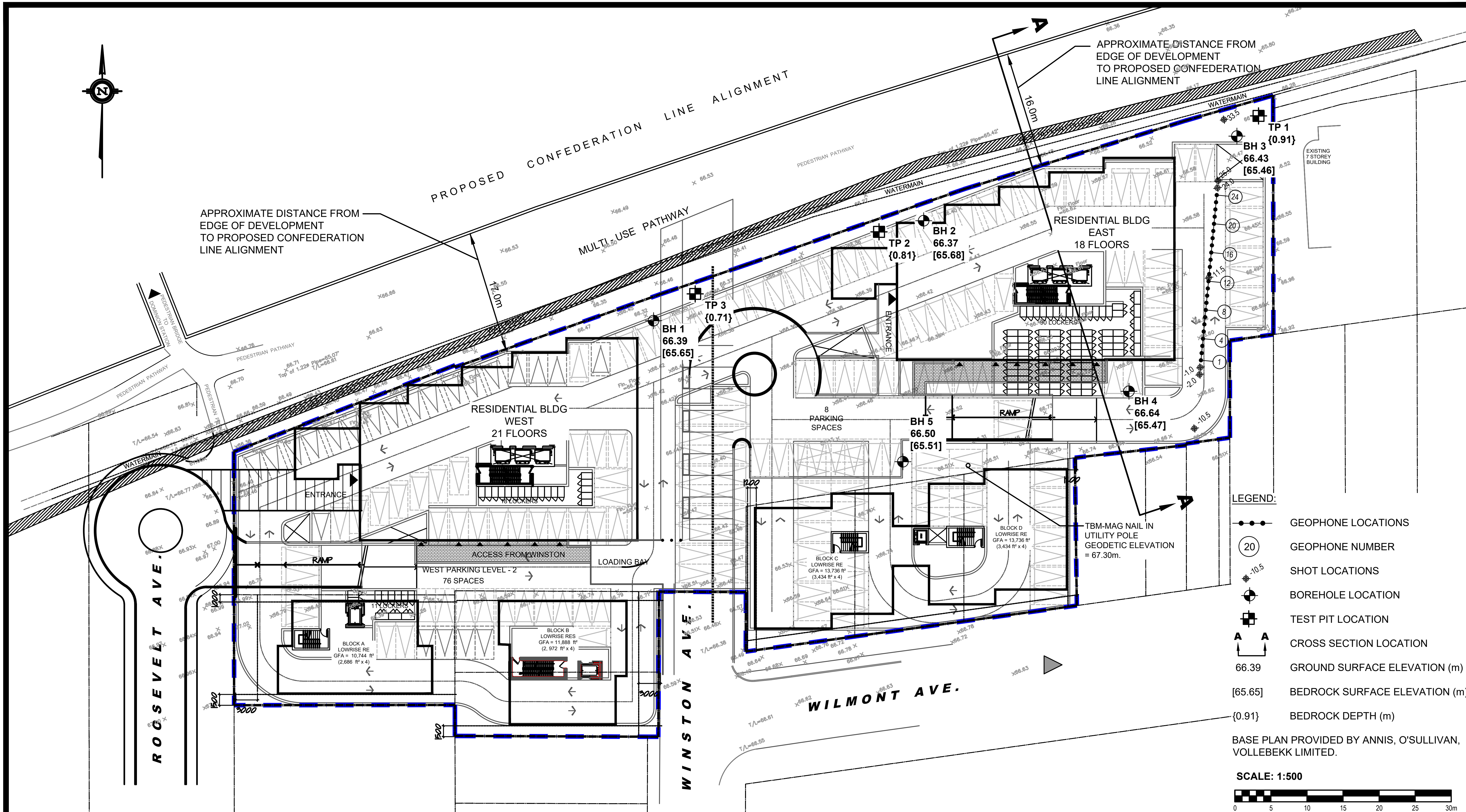
APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG2178-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN



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NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO LATEST CONCEPTUAL PLAN	07/09/2020	DP

UNIFORM URBAN DEVELOPMENTS

GEOTECHNICAL INVESTIGATION - PROPOSED RESIDENTIAL DEVELOPMENT
335 ROOSEVELT AVENUE

OTTAWA, ONTARIO

Title: **TEST HOLE LOCATION PLAN**

Scale: 1:500	Date: 06/2020
Drawn by: RCG	Report No.: PG2178
Checked by: DP	**PG2178-1**
Approved by: DJG	Revision No.: 1

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