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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Supplemental Geotechnical Investigation

Proposed Hi-Rise Development
Clyde Avenue at Baseline Road
Ottawa, Ontario

Prepared For

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Report: PG4871-1

Table of Contents

Appendices

Appendix 1 Soil Profile and Test Data Sheets
Symbols and Terms
Analytical Testing Results

Appendix 2 Figure 1 - Key Plan
Shear Wave Velocity Measurement Report
Drawing PG4871-1 - Test Hole Location Plan
Drawing PG4871-2 - Seismic Survey Location

1.0 Introduction

Paterson Group (Paterson) was commissioned by Groupe Sélection to conduct a supplemental geotechnical investigation for the proposed high rise development to be located at Clyde Avenue and Baseline Road in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the current investigation was to:

❏ Determine the subsoil and groundwater conditions at this site by means of boreholes.

❏ Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

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

2.0 Proposed Development

The proposed development is understood to consist of a 13 storey building with 2 levels of underground parking. The underground parking levels are anticipated to extend beyond the proposed building footprint and occupy the majority of the subject site. At-grade paved access lanes, parking areas and landscaped areas are also anticipated.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was completed on March 25, 2019. At that time, 4 boreholes were advanced to a maximum depth of 7.6 m below existing grade. Previous investigations were carried out on June 11 to 13, 2008 and August 17, 2009. The borehole locations were distributed in a manner to provide general coverage of the proposed development. The locations were determined in the field by Paterson personnel taking into consideration site features and underground services. The locations of the test holes are shown in Drawing PG4871-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were advanced using a truck-mounted auger drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from the geotechnical department. The drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

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

Coring of the bedrock using diamond drilling was carried out at two borehole locations to assess the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one drilled section over the length of the drilled section. These values are indicative of the quality of the bedrock.

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 of this report.

Groundwater

Flexible standpipes were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The test hole locations were selected in the field by Paterson personnel in a manner to provide general coverage of the subject site, taking into consideration site features. The ground surface elevations at the borehole locations were provided by Stantec Geomatics. The test hole locations and ground surface elevations at the test hole locations are presented in Drawing PG4871-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

The subject site is currently unoccupied vacant land with a grass surface. During winter months the subject site is used as overflow parking for the neighbouring commercial properties and businesses. The site is relatively flat and approximately at grade with neighbouring properties and nearby roadways. The site is bordered to the north and east by commercial properties, to the south by Baseline Road and to the west by Clyde Avenue.

4.2 Subsurface Profile

Generally, the soil profile at the borehole locations consisted of a thin layer of topsoil and/or gravel fill followed by a layer of silty sand. A glacial till deposit was encountered below the fill/silty sand layer. The fine matrix of the glacial till was observed to consist of silty sand with gravel, cobbles and trace clay. Practical auger refusal was encountered at depths ranging from 2.6 to 3.5 m below ground surface. Specific details of the soil profile at each borehole location are provided on the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Grey limestone bedrock was cored at BHs 2 and 3 to a maximum depth of 7.7 m. The recovery values and RQD values for the bedrock cores were calculated. The recorded recovery values were 100% for all samples, while the RQD values vary between 82 and 100%. Based on these results the quality of the bedrock ranges from good to excellent.

Based on available geological mapping, the local bedrock consists of interbedded limestone and dolostone of the Gull River formation with an anticipated overburden thickness of 0 to 1 m.

4.3 Groundwater

Groundwater levels (GWL) were measured on April 1, 2019 in the standpipes installed at the borehole locations. The GWL readings and open hole groundwater observations are presented in Table 1 below. It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole. The groundwater table level can also be estimated based on moisture levels and colour of the recovered soil samples. Based on these observations at the borehole locations, the permanent groundwater table is expected below a 3 m depth. As groundwater levels are subject to seasonal fluctuations, it should be noted that groundwater may vary at the time of construction.

Table 1 - Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level (m)		Recording Date
		Depth	Groundwater Elevation (m)	
BH 1	98.37	Blocked	-	April 1, 2019
BH 2	98.92	3.67	95.25	April 1, 2019
BH 3	98.37	2.11	96.26	April 1, 2019
BH 4	98.54	2.22	96.32	April 1, 2019

Note: The ground surface elevations at the borehole locations were provided by Stantec Geomatics.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed residential hi-rise development. It is anticipated that bedrock removal will be required to accommodate the proposed underground parking levels.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings and other settlement sensitive structures. Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only a small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or 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 can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing.

Fill Placement

Fill used for grading purposes 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. An OPSS Granular B Type I (pit run) material or existing granular material approved by the geotechnical consultant can also be used below the proposed building footprint. The fill should be tested and approved prior to delivery to the site. It should be placed in lifts no greater than 300 mm in thickness and compacted using suitable compaction equipment for the specified lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of its 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. These materials should be spread in thin lifts and be compacted at minimum 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 SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls, unless deemed acceptable by the geotechnical consultant.

5.3 Foundation Design

Shallow footings placed on a clean, surface sounded bedrock bearing surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5. Footings placed on a clean, surface-sounded bedrock bearing surface will be subjected to negligible post-construction total and differential settlements.

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.

For the footings at depth for the parking garage and building foundation, a factored bearing resistance value at ULS of **4,500 kPa**, incorporating a geotechnical resistance factor of 0.5 could be used if founded on limestone bedrock provided the bedrock is free of seams, fractures and voids within 1.5 m below the founding level.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support. Near vertical (1H:6V) slopes can be used for unfractured bedrock bearing media; a 1H:1V slope can be used for fractured or weathered bedrock.

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 buildings from Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel and the interpretation was completed by Dr. Dariush Motazedian, an expert seismologist with Carleton University. The results of the shear wave interpretation are presented in Appendix 2.

Field Program

The shear wave testing was located adjacent to the subject site, as presented in Drawing PG4871-2 - Seismic Survey Location Plan presented in Appendix 2. 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 a 75 mm ground spike attached to the geophone land case. The geophones were spaced at 3 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 4 to 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 3, 4.5 and 30 m away from the first and last geophone.

The methods of testing completed by Paterson are guided by the standard testing procedures used by the expert seismologists at Carleton University and Geological Survey of Canada (GSC).

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Dr. Dariush Motazedian, an expert seismologist with Carleton University. The 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. 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, this 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.

Based on the available concept drawings and plans, the building footings will be placed directly on the bedrock.

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

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

$$V_{s30} = \frac{30m}{\sum\left(\frac{30m}{2{,}907m/s}\right)}$$

$$V_{s30} = 2{,}907m/s$$

Based on the results of the seismic testing , the average shear wave velocity of the upper 30 m profile below the proposed underside of foundation, Vs_{30}, was calculated to be **2,907 m/s**. Therefore, a **Site Class A** is applicable for the proposed buildings founded directly on bedrock as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the site are not susceptible to liquefaction.

5.5 Basement Slab

All overburden soil will be removed for the proposed building and underground parking levels and the basement floor slab will be placed over a bedrock medium. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consist of a 19 mm clear crushed stone.

In consideration of the groundwater conditions encountered during the investigation, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor.

5.6 Basement Wall

Where soil is to be retained, 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 dry unit weight of 20 kN/m^3. Undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective 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.

Two distinct conditions, static and seismic should be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

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

Generally, the unconfined compressive strength of limestone and dolostone bedrock exceeds 80 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of 1,000 kPa, 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 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, 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 calculations the parameters given in Table 2 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) - Good Quality Limestone Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	80 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 Design

Car only parking and heavy traffic areas are anticipated at this site. The subgrade material will consist of glacial till. The proposed pavement structures are shown in Tables 4 and 5.

Table 4 - 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 5 - Recommended Pavement Structure - Access Lanes and Heavy Truck Parking Areas

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 I or Type II material.

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

Pavement Structure Drainage

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

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. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. It is anticipated that imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. It should also be noted that existing granular material approved by the geotechnical consultant is also acceptable for backfill against the foundation walls.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard.

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other exterior unheated footings.

For footings founded directly on sound bedrock where insufficient soil cover is available, insulation can be used to make up the shortfall. Also, if the bedrock is considered to be non-frost susceptible, the suggested insulation can be omitted. Non-frost susceptible bedrock requires confirmation of no significant soil bearing seams within the potential frost penetration depth. This confirmation is carried out using probeholes which are verified by geotechnical personnel.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be 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.

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

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. 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 225 mm thick lifts compacted to a minimum of 95% of the material’s SPMDD.

Generally, it should be possible to re-use the moist, not wet, glacial till above the cover material if the excavation and filling operations are carried out in dry weather conditions. The wet glacial till should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

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 material’s SPMDD.

6.5 Groundwater Control

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

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

For typical ground or surface water volumes being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register in 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.

6.6 Winter Construction

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

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the 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 the footings are protected with sufficient soil cover to prevent freezing at founding level.

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

6.7 Corrosion Potential and Sulphate

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

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 geotechnical aspects of the excavation contractor's shoring design, prior to construction.
- ❑ Review the bedrock stabilization and excavation requirements.
- ❑ Review proposed foundation drainage design and requirements.
- ❑ 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 and follow-up 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. We request that we be permitted to review the grading plan once available. Also, our recommendations should be reviewed when the project drawings and specifications are complete.

A 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 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 Groupe Sélection 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.

Nathan F. S. Christie, P.Eng.

Carlos P. Da Silva, P.Eng., ing, QP_{ESA}

LICENSED PROFESSIONAL ENGINEER
Apr 3-2019
C. P. DA SILVA
PROVINCE OF ONTARIO

Report Distribution:

- ❑ Groupe Sélection (3 copies)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Clyde Avenue and Baseline Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger **DATE** March 25, 2019

FILE NO. PG4871

HOLE NO. BH 1

SOIL DESCRIPTION	Depth of strata (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.37
FILL: Gravel	0.08	AU	1				
Compact, brown **SILTY SAND**	1.52	SS	2	75	14	1	97.37
GLACIAL TILL: Compact to dense, brown silty sand with gravel, some cobbles, trace clay	2.59	SS	3	83	14	2	96.37
		SS	4	92	50+		

End of Borehole

Practical refusal to augering at 2.59m depth

(Piezometer blocked at 1.3m depth - April 1, 2019)

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

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Clyde Avenue and Baseline Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 25, 2019

FILE NO. PG4871

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	98.92
FILL: Brown silty sand with gravel	0.53	AU	1				
Loose, brown **SILTY SAND**	1.37	SS	2	54	9	1	97.92
GLACIAL TILL: Compact to dense, brown silty sand with gravel, some cobbles	3.02	SS	3	83	12	2	96.92
		SS	4	88	33	3	95.92
BEDROCK: Good to excellent quality, grey limestone	7.11	RC	1	100	95	4	94.92
		RC	2	100	90	5	93.92
		RC	3	100	82	6	92.92
End of Borehole						7	91.92

(GWL @ 3.67m - April 1, 2019)

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

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Clyde Avenue and Baseline Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 25, 2019

FILE NO. PG4871

HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	98.37
TOPSOIL	0.10	AU	1				
Very loose to loose, brown **SILTY SAND**	1.60	SS	2	75	1	1	97.37
GLACIAL TILL: Compact to dense, brown silty sand with gravel, some cobbles, trace clay	3.30	SS	3	54	9	2	96.37
		SS	4	71	26	3	95.37
		SS	5	75	50+		
BEDROCK: Good to excellent quality, grey limestone	7.67	RC	1	100	91	4	94.37
		RC	2	100	98	5	93.37
		RC	3	100	83	6	92.37
		RC	4	100	100	7	91.37
End of Borehole							
(GWL @ 2.11m - April 1, 2019)							

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

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Clyde Avenue and Baseline Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE March 25, 2019

FILE NO. PG4871

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	98.54
TOPSOIL	0.30	AU	1				
Very loose, brown **SILTY SAND**	1.30	SS	2	58	3	1	97.54
Brown **SILTY CLAY**	1.37						
GLACIAL TILL: Compact to dense, brown silty sand with gravel, trace clay		SS	3	33	6	2	96.54
		SS	4	67	16	3	95.54
	3.45	SS	5	100	50+		

End of Borehole

Practical refusal to augering at 3.45m depth

(GWL @ 2.22m - April 1, 2019)

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

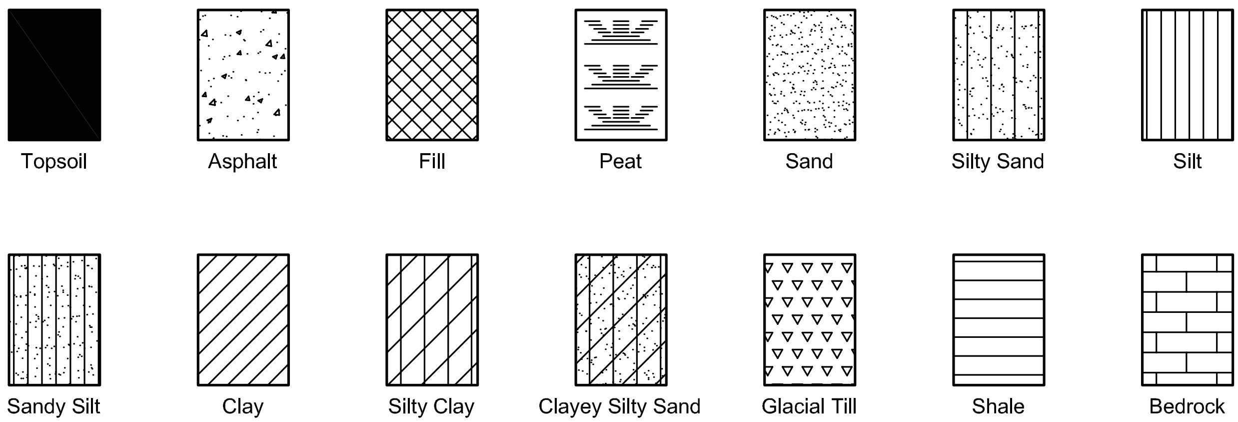
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

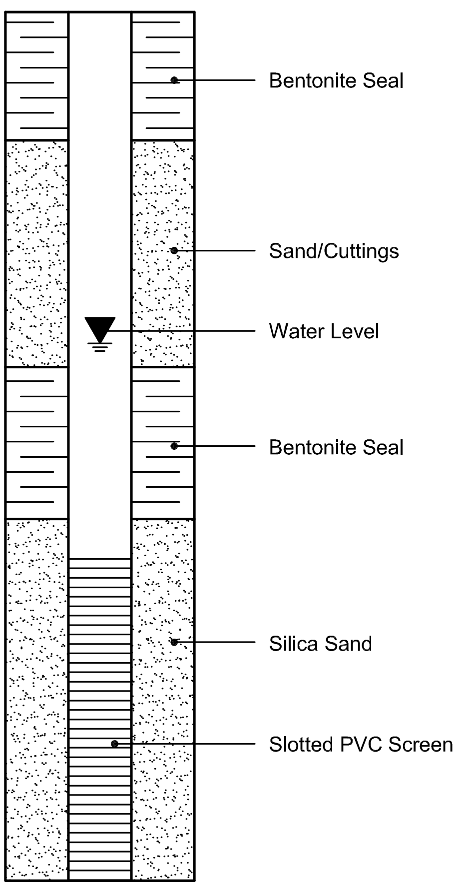
SYMBOLS AND TERMS (continued)

STRATA PLOT

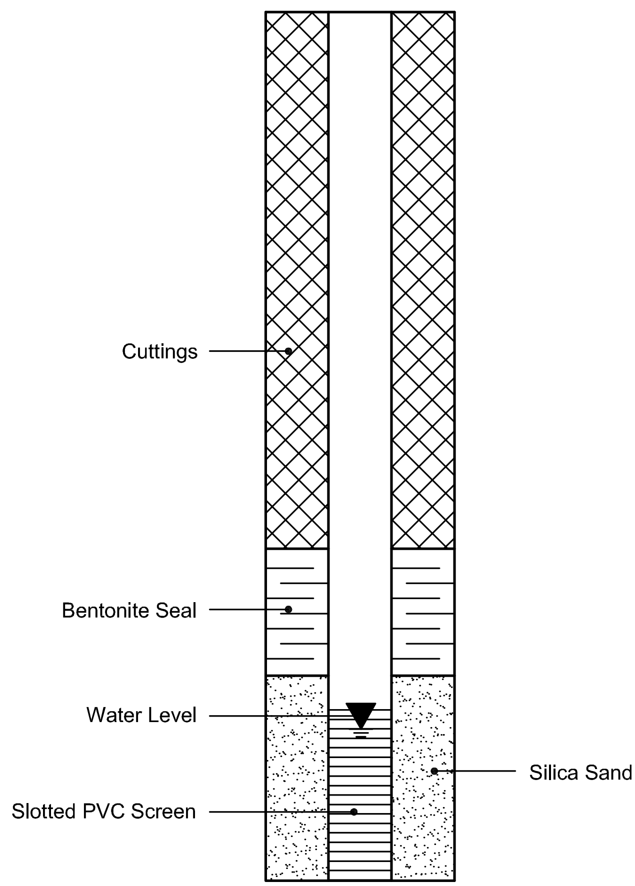


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 26252

Report Date: 29-Mar-2019

Order Date: 25-Mar-2019

Project Description: PG4871

	Client ID:	BH3-SS2	-	-	-
	Sample Date:	03/25/2019 12:00	-	-	-
	Sample ID:	1913155-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	81.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.38	-	-	-
Resistivity	0.10 Ohm.m	18.2	-	-	-
Anions					
Chloride	5 ug/g dry	274	-	-	-
Sulphate	5 ug/g dry	26	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

SHEAR WAVE VELOCITY MEASUREMENT REPORT

DRAWING PG4871-1 - TEST HOLE LOCATION PLAN

DRAWING PG4871-2 - SEISMIC SURVEY LOCATION

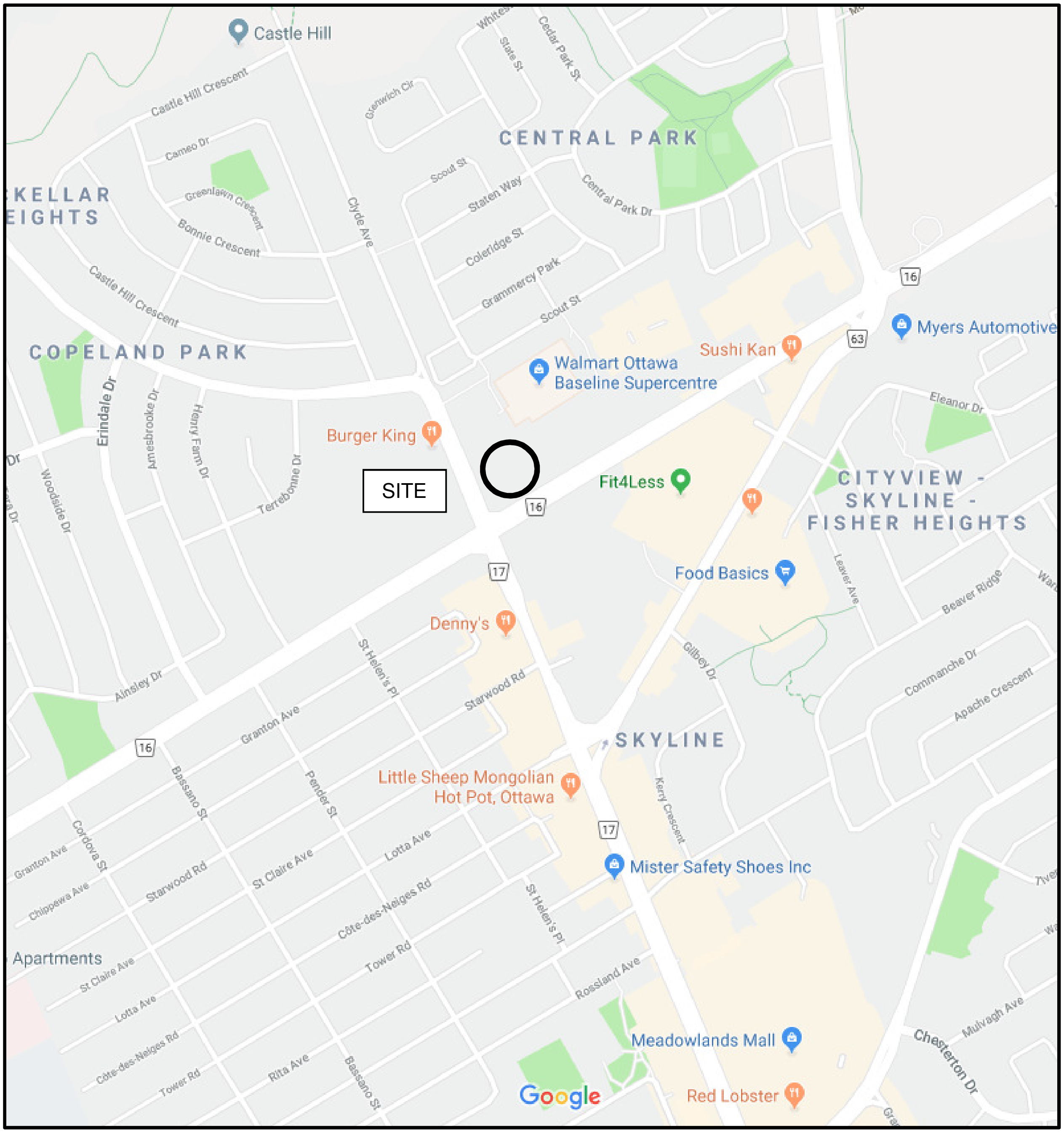


FIGURE 1

KEY PLAN

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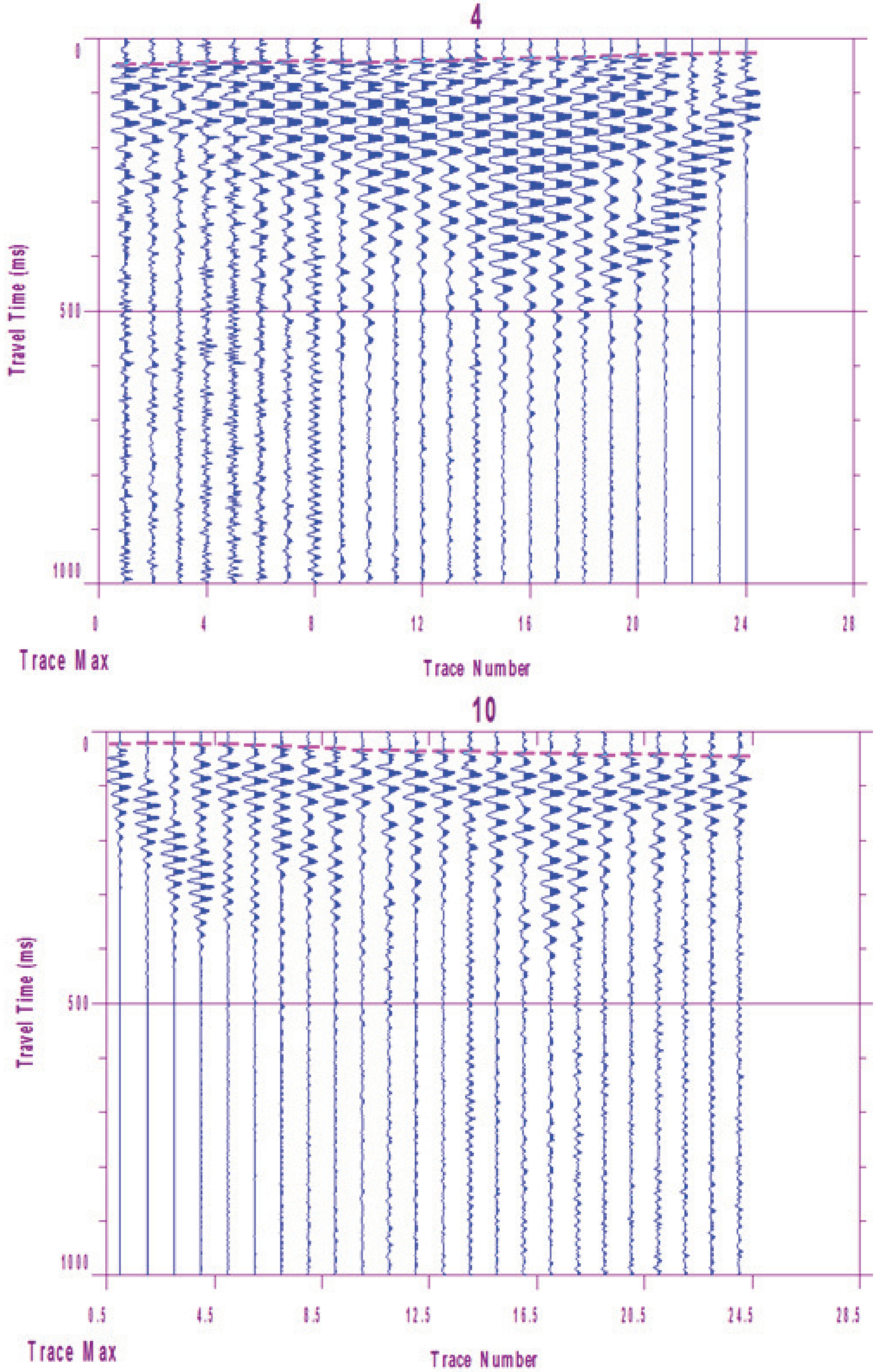
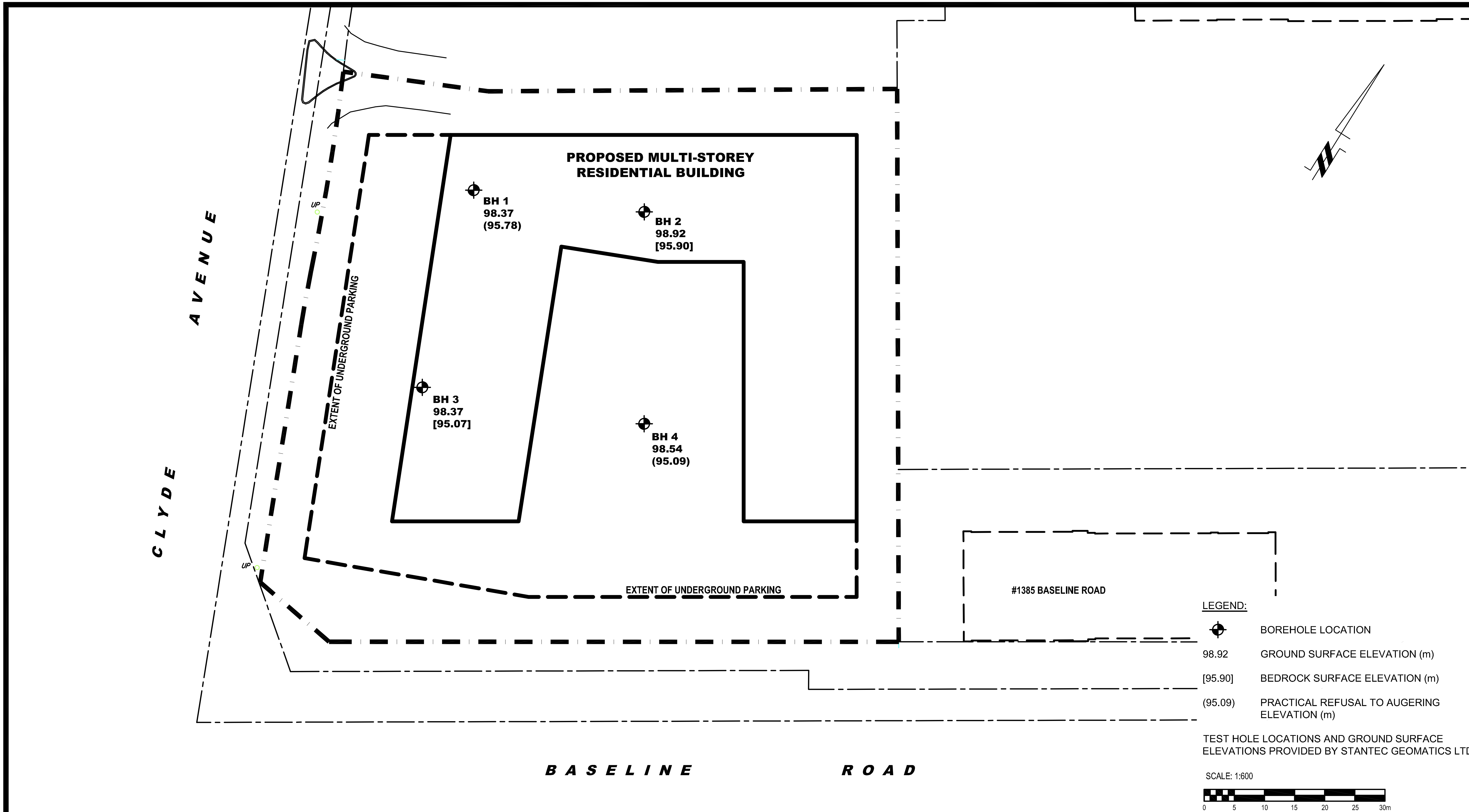
Figure 1. A few raw records, as examples, at -4.5m (graph #4) and +3m (graph #10).





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NO.	REVISIONS	DATE	INITIAL

GROUPE SELECTION

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDING - CLYDE AVE. AT BASELINE ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:600	Date: 03/2019
Drawn by: MPG	Report No.: PG4871-1
Checked by: NC	PG4871-1
Approved by: DJG	Revision No.:

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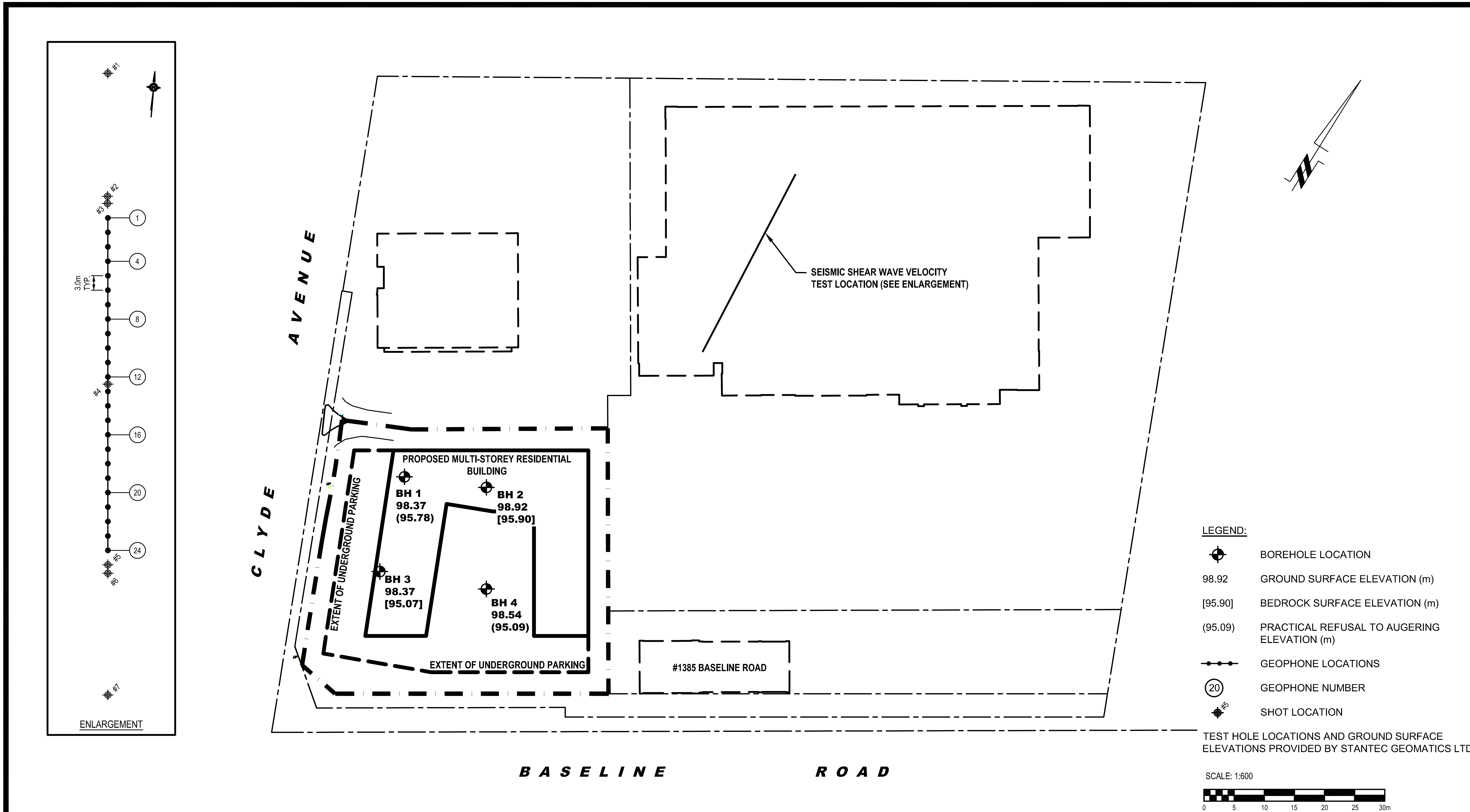

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GROUPE SELECTION

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDING - CLYDE AVE. AT BASELINE ROAD
OTTAWA, ONTARIO

Title:
SEISMIC SURVEY LOCATION

Scale: 1:1250	Date: 03/2019
Drawn by: MPG	Report No.: PG4871-1
Checked by: NC	**PG4871-2**
Approved by: DJG	Revision No.:

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