paterson group

Geotechnical Investigation
Proposed Fire Station
1075 March Road - Ottawa, Ontario

Prepared For

City of Ottawa

January 20, 2021

Report PG5321-1 Revision 1

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Appendices

1.0   Introduction

Paterson Group (Paterson) was commissioned by the City of Ottawa to conduct a geotechnical investigation for the proposed Fire Station to be located within the Kanata North Development at 1045 March Road, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the investigation were to:

❏   Determine the subsurface soil and groundwater conditions 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.   This report contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as understood at the time of writing this report.

2.0   Proposed Project

Based on preliminary design details, it is understood that a three-bay fire station is proposed along with an office building. Associated access lanes, parking areas and landscaped areas are expected for this project. It is furthermore, expected that the building will be municipally serviced.

3.0   Method of Investigation

3.1   Field Investigation

Field Program

The field program for the current investigation was carried out on April 27, 2020. At that time, 4 boreholes were completed to a maximum depth of 4.3 m below existing ground surface. The test hole locations were placed in a manner to provide general coverage of the subject site taking into consideration site features and underground utilities. The test hole locations for the current investigation are presented on Drawing PG5321-1 - Test Hole Location Plan included in Appendix 2.

A previous investigation that was conducted on this site, several test pits were conducted in the general area including one on the proposed parcel. The previous field investigation was conducted on March 21, 2013.

The boreholes were completed using a track-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 from the geotechnical division. The testing procedure consisted of augering to the required depths and at the selected locations sampling the overburden.

Sampling and In Situ Testing

Soil samples collected from the boreholes were either recovered directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Soil samples from the test pits were recovered from the side walls of the open excavation. All soil samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags and transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and grab samples were recovered from the boreholes are shown as AU, SS and G, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils. Subsurface conditions observed in the test holes were recorded in detail in the field. Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profile encountered at the test hole locations.

Groundwater

Monitoring wells were installed in the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

3.2   Field Survey

The test hole locations were selected by Paterson personnel in a manner to provide general coverage of the proposed building, taking into consideration site features.

The borehole locations and ground surface elevations were surveyed by Paterson and the ground surface elevations are referenced to a geodetic datum. The test hole locations are presented on Drawing PG5321-1 - 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. Soil samples will be stored for a period of one month after this report is completed, unless otherwise directed.

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.

4.0 Observations

4.1 Surface Conditions

The subject site is currently undeveloped and used as agricultural land. The ground surface is generally flat and gently slopes down from west to east towards March Road. An existing creek flows from west to east south the subject site. The site is bordered to the south by a residential dwelling and agricultural land, to the west by agricultural land and to the north by an elementary school.

4.2 Subsurface Profile

Overburden

Generally, the soil profile encountered at the test hole locations consists of an agriculturally disturbed clayey organic layer overlying a hard to very stiff brown silty clay crust. Refusal to augering was encountered at all borehole locations between 3.4 and 4.3 m. A thin layer of glacial till composed of sand, gravel and cobbles in a silty clay soil matrix was encountered in BH1.   Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Based on available geological mapping, the bedrock in the area is part of the March formation, which consists of sandstone and dolomite . Also, based on available geological mapping, the overburden thickness is expected to range from 3 to 5 m.

Silty Clay

Silty clay was encountered immediately beneath the topsoil at all test hole locations. The silty clay layer has been weathered to a hard to very stiff brown crust. The crust extends to depths varying between 3.4 and 4.3 m. In situ shear vane field testing carried out within the silty clay layer in the lower portion of the weathered crust yielded undrained shear strength values ranging from approximately 120 to over 250 kPa. These values are indicative of a very stiff to hard consistency.

4.3   Groundwater

The groundwater level (GWL) readings were recorded at the borehole locations on May 1, 2020 and are presented on the Soil Profile and Test Data sheets. It is important to note that based on observations of the soil samples recovered from the borehole locations, such as colouring, moisture levels and consistency, the long-term groundwater level is not expected within the overburden soils. The groundwater level readings from nearby monitoring wells indicate that an artesian pressure may be present below the bedrock surface. It should be noted that groundwater levels are subject to seasonal fluctuations and therefore groundwater levels could differ at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed fire station. It is expected that the proposed building will be founded on an undisturbed, very stiff silty clay bearing surface or over the bedrock surface by means of a zero entry, vertical trench in-filled with lean concrete to design underside of footing level.

Bedrock removal will most likely not be required to complete the fire station excavation based on the current design details. Moderate to low groundwater infiltration through the excavated clay is expected during construction of the building.

Due to the presence of the sensitive silty clay layer, a proposed development will be subjected to grade raise restrictions.

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

5.2 Site Preparation

Stripping Depth

Topsoil and deleterious materials, such as those containing significant amounts of organics, should be removed from within any settlement sensitive structure.

Fill Placement

Fill used for grading beneath any settlement sensitive structures should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the proposed 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. In landscaped areas, 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 SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Lean Concrete Filled Trenches

Where the proposed footings are to be founded on bedrock which is located below the underside of footing elevation, zero-entry vertical trenches should be excavated to the clean, surface sounded bedrock, and backfilled with lean concrete to the founding elevation (minimum 17 MPa 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3   Bearing Resistance Values

Footings placed on an undisturbed, very stiff silty clay bearing surface or engineered fill placed over an undisturbed, very stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **350 kPa.**

Footings placed over a lean concrete in-filled trench as detailed above, which extends down to a clean, weathered bedrock surface can be designed using a bearing resistance value at ULS of **1,500 kPa.** A clean, weathered bedrock surface consists of one from which all topsoil, soils, deleterious materials and loose rock have been removed prior to concrete placement.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

The above noted allowable bearing capacities are provided for design purposes and should be confirmed in the field prior to placement of concrete for structures.

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 engineered fill or native soil above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

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).

Permissible Grade Raise Recommendations

Based on the undrained shear strength testing results and experience with the local silty clay deposit. It is recommended that a permissible grade raise restriction of 3.0 m be implemented for the subject site.

5.4   Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012. The results of the shear wave velocity testing are attached to the present report.

Field Program

The seismic array location is presented on Drawing PG5321-1 - Test Hole Location Plan presented in Appendix 2. Paterson field personnel placed 18 horizontal geophones in a straight line in roughly an east-west 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 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 3, 4.5 and 25 m away from the first geophone and last geophones.

Data Processing and Interpretation

Interpretation of the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile immediately below the proposed building foundations. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

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

Based on our testing results, the average overburden shear wave velocity is 181 m/s, while the bedrock shear wave velocity is 2,615 m/s. Considering the shallow depth to bedrock encountered throughout the subject site, it is not anticipated that more than 3 m of overburden soils will be present below the buildings underside of footing (USF). Assuming up to 3 m of overburden soils may be present between USF and bedrock, the Vs_{30} calculated using the standard equation for average shear wave velocity provided in OBC 2012, applies as presented below:

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

$$V_{s30} = \frac{30m}{\left(\frac{3m}{181m/s} + \frac{27m}{2,615m/s} \right)}$$

$$V_{s30} = 1,115m/s$$

Based on the results of the seismic shear wave velocity testing, the average shear wave velocity, V_{s30}, for the proposed building is 1,115 m/s for up to 3 m of overburden soils present between USF and bedrock. **Based on these assumptions, a Site Class B is applicable for design of the proposed building.**

However, if the footings are supported on near vertical lean-concrete in-filled trenches which extend through the overburden soils to an approved bedrock bearing surface, a Site Class A is applicable for design of the proposed building, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Slab-on-Grade Construction / Basement Slab

With the removal of all topsoil and deleterious fill from within the footprint of the proposed buildings, the native soil surface or approved engineered fill surface will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. For structures with basement slabs, it is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone.

For any structures with slab-on-grade construction, the upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, in our opinion, 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. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

The total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component (ΔP_{AE}).

Static Earth Pressures

The static horizontal earth pressure (P_o) can be calculated by 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)

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) can be calculated using the earth pressure distribution equal to $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ 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. The vertical seismic coefficient is assumed to be zero.

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

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

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

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

5.7   Pavement Structure

For design purposes, the pavement structure presented in Table 2 is recommended for fire station access lanes and car parking area.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas	
Thickness (mm)	**Material Description**
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 3 - Recommended Pavement Structure - Access Lanes 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
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, such as Terratrack 200 or equivalent, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

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

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on 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 load carrying capacity.

Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0   Design and Construction Precautions

6.1   Foundation Drainage and Backfill

A perimeter foundation drainage system is recommended for proposed structures. The system should consist of a 150 mm diameter, geotextile-wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. The site materials will be frost susceptible and, as such, are not recommended for re-use as backfill unless a composite drainage system (such as system Platon or Miradrain G100N) connected to a drainage system is provided.

6.2   Protection Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 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 exterior unheated footings, not thermally connected to a heated space, such as exterior columns and/or wing walls.

6.3   Excavation Side Slopes

The excavations for the proposed development will be mostly through a stiff silty clay. Where excavation is above the groundwater level to a depth of approximately 3 m, the excavation side slopes should be stable in the short term at 1H:1V. Flatter slopes could be required for deeper excavations or for excavation below the groundwater level. Where such side slopes are not permissible or practical, temporary shoring should be used. The subsoil at this site is considered to be mainly a Type 2 or 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

The slope cross-sections recommended above are for temporary slopes. 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.

It is expected that deep service trenches in excess of 3 m will be completed using a temporary shoring system designed by a structural engineer, such as stacked trench boxes in conjunction with steel plates. The trench boxes should be installed to ensure that the excavation sidewalls are tight to the outside of the trench boxes and that the steel plates are extended below the base of the excavation to prevent basal heave (if required).

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.

6.4   Pipe Bedding and Backfill

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 a minimum of 300 mm above the pipe obvert should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% SPMDD.

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

6.5   Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be moderate to low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of the shallow excavation. 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.

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

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

6.6   Winter Construction

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

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

Trench excavations and pavement construction are difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be 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 a low corrosive environment.

7.0   Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

❏   Review of the finalized fire station design drawings from a geotechnical perspective, once available.

❏   Periodic site visits during controlled blasting operations and to monitoring the groundwater influx during construction.

❏   Observation of all bearing surfaces prior to the placement of concrete and/or precast structures.

❏   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 and bearing surfaces prior to backfilling.

❏   Field 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 request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0   Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request permission to review the grading plan once available. Also, our recommendations should be reviewed when the drawings and specifications are complete.

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

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 The City of Ottawa or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

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

Jan 20, 2021

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Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TEST RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Fire Station 45 - 1045 March Road
Ottawa, Ontario

DATUM Geodetic	**FILE NO.** **PG5321**
REMARKS	**HOLE NO.** **BH 1**
BORINGS BY CME-55 Low Clearance Drill	**DATE** April 27, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD				
GROUND SURFACE						0	83.23		
TOPSOIL 0.15									
		AU	1						
		SS	2	92	6	1	82.23		
Very stiff, brown **SILTY CLAY**		SS	3	100	6	2	81.23		
								121 ▲	
						3	80.23		
								121 ▲	
3.80									
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders		SS	4	80	50+	4	79.23		
4.19									
End of Borehole									
Practical refusal to augering at 4.19m depth									
(GWL @ 0.04m - May 1, 2020)									

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Fire Station 45 - 1045 March Road
Ottawa, Ontario

DATUM	Geodetic	FILE NO. **PG5321**

REMARKS		HOLE NO. **BH 2**

BORINGS BY	CME-55 Low Clearance Drill	DATE April 27, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / Water Content %	Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD				
GROUND SURFACE						0	83.85		
TOPSOIL 0.10									
		AU	1						
Hard to very stiff, brown **SILTY CLAY**		SS	2	100	9	1	82.85		▼
		SS	3	100	7				
						2	81.85		
								250 ▲	
- trace gravel by 3.0m depth						3	80.85		
3.40								144 ▲	
End of Borehole									
Practical refusal to augering at 3.40m depth									
(GWL @ 1.17m - May 1, 2020)									

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

○ Water Content %
20 40 60 80

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Fire Station 45 - 1045 March Road
Ottawa, Ontario

DATUM Geodetic	**FILE NO.** **PG5321**
REMARKS	**HOLE NO.** **BH 3**
BORINGS BY CME-55 Low Clearance Drill **DATE** April 27, 2020	

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / Water Content %	Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD				
GROUND SURFACE						0 — 82.97			
TOPSOIL 0.13									
		AU	1						
		SS	2	83	9	1 — 81.97			
Hard, brown **SILTY CLAY**		SS	3	100	7	2 — 80.97			
						3 — 79.97		250 ▲	
								229 ▲	
4.04						4 — 78.97		209 ▲	

End of Borehole

Practical refusal to augering at 4.04m depth

(GWL @ 0.31m - May 1, 2020)

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

○ Water Content %
20 40 60 80

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Fire Station 45 - 1045 March Road
Ottawa, Ontario

DATUM Geodetic	**FILE NO.** **PG5321**
REMARKS	**HOLE NO.** **BH 4**
BORINGS BY CME-55 Low Clearance Drill **DATE** April 27, 2020	

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD				
GROUND SURFACE						0 — 82.33			
TOPSOIL 0.13									
		AU	1						
		SS	2	100	13	1 — 81.33			
Hard to very stiff, brown **SILTY CLAY**		SS	3	100	9	2 — 80.33			
								250 ▲	
						3 — 79.33			
								△ 249 ▲	
						4 — 78.33			120 ▲
4.27									
End of Borehole									
Practical refusal to augering at 4.27m depth									
(GWL @ 0.03m - May 1, 2020)									

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

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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Future Development Lands - March Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.	**FILE NO.** PG2878
REMARKS 18T 0425826; 5024040	
BORINGS BY Hydraulic Excavator **DATE** March 21, 2013	**HOLE NO.** TP35

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY %	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / Water Content %	Piezometer Construction
GROUND SURFACE						0	82.99		
TOPSOIL									
0.46									
Hard to very stiff, brown **SILTY CLAY**		G	1			1	81.99		
						2	80.99		
						3	79.99		
						4	78.99		
4.27		G	2						
End of Test Pit Practical refusal to excavation on inferred bedrock surface at 4.27m depth

(GWL @ 2.7m depth based on field observations) | | | | | | | | | |

Water Content %
20   40   60   80

Shear Strength (kPa)
20   40   60   80   100

▲ 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 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

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.

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 = $(D30)^2 / (D10 \times D60)$
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

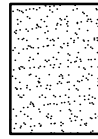|		
k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during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

SYMBOLS AND TERMS (continued)

STRATA PLOT
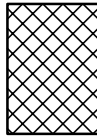
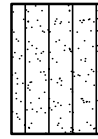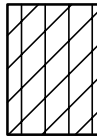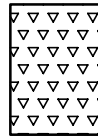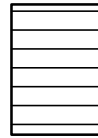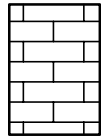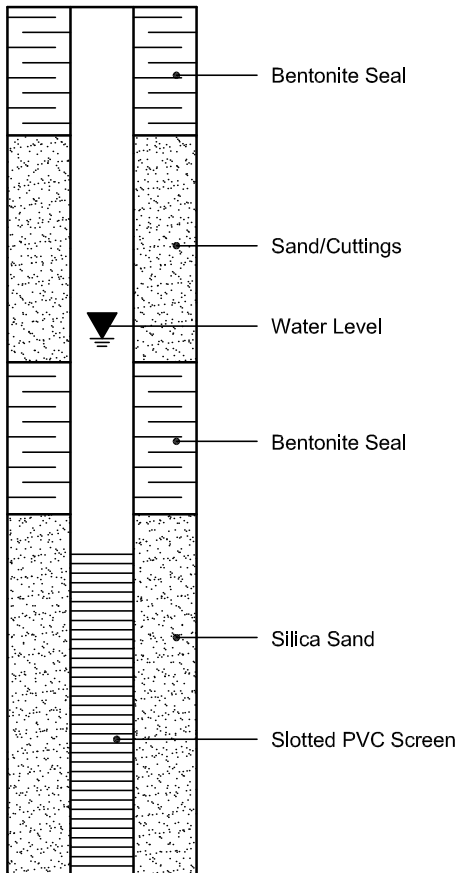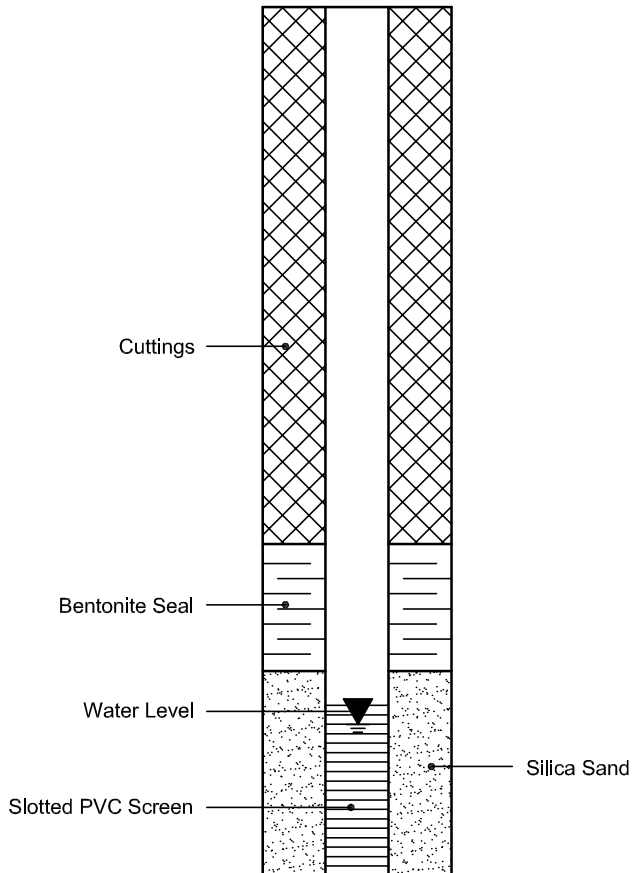|						
Topsoil	Asphalt	Fill	Peat	Sand	Silty Sand	Silt
Sandy Silt	Clay	Silty Clay	Clayey Silty Sand	Glacial Till	Shale	Bedrock
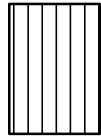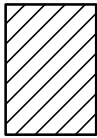
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION

- Bentonite Seal
- Sand/Cuttings
- Water Level
- Bentonite Seal
- Silica Sand
- Slotted PVC Screen

PIEZOMETER CONSTRUCTION

- Cuttings
- Bentonite Seal
- Water Level
- Slotted PVC Screen
- Silica Sand

PARACEL
LABORATORIES LTD.

Certificate of Analysis

Report Date: 05-May-2020

Client: Paterson Group Consulting Engineers

Order Date: 29-Apr-2020

Client PO: 30015

Project Description: PG5321

Client ID:		BH2-SS3	-	-	-
Sample Date:		27-Apr-20 13:00	-	-	-
Sample ID:		2018207-01	-	-	-
MDL/Units		Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	74.8	-	-	-
General Inorganics					
pH	0.05 pH Units	6.83	-	-	-
Resistivity	0.10 Ohm.m	103	-	-	-
Anions					
Chloride	5 ug/g dry	10	-	-	-
Sulphate	5 ug/g dry	7	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 and 3 - SEISMIC SHEAR WAVE PROFILES

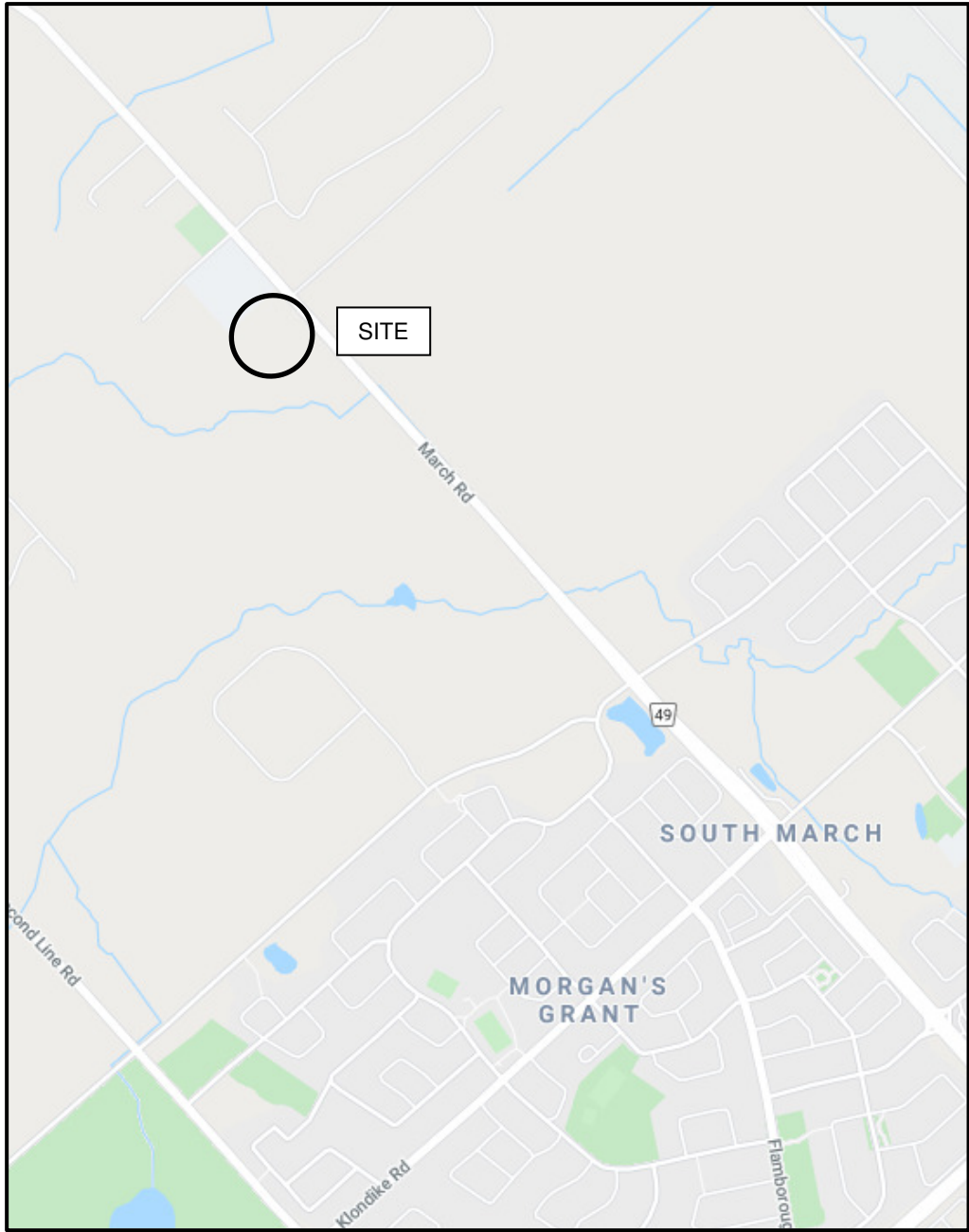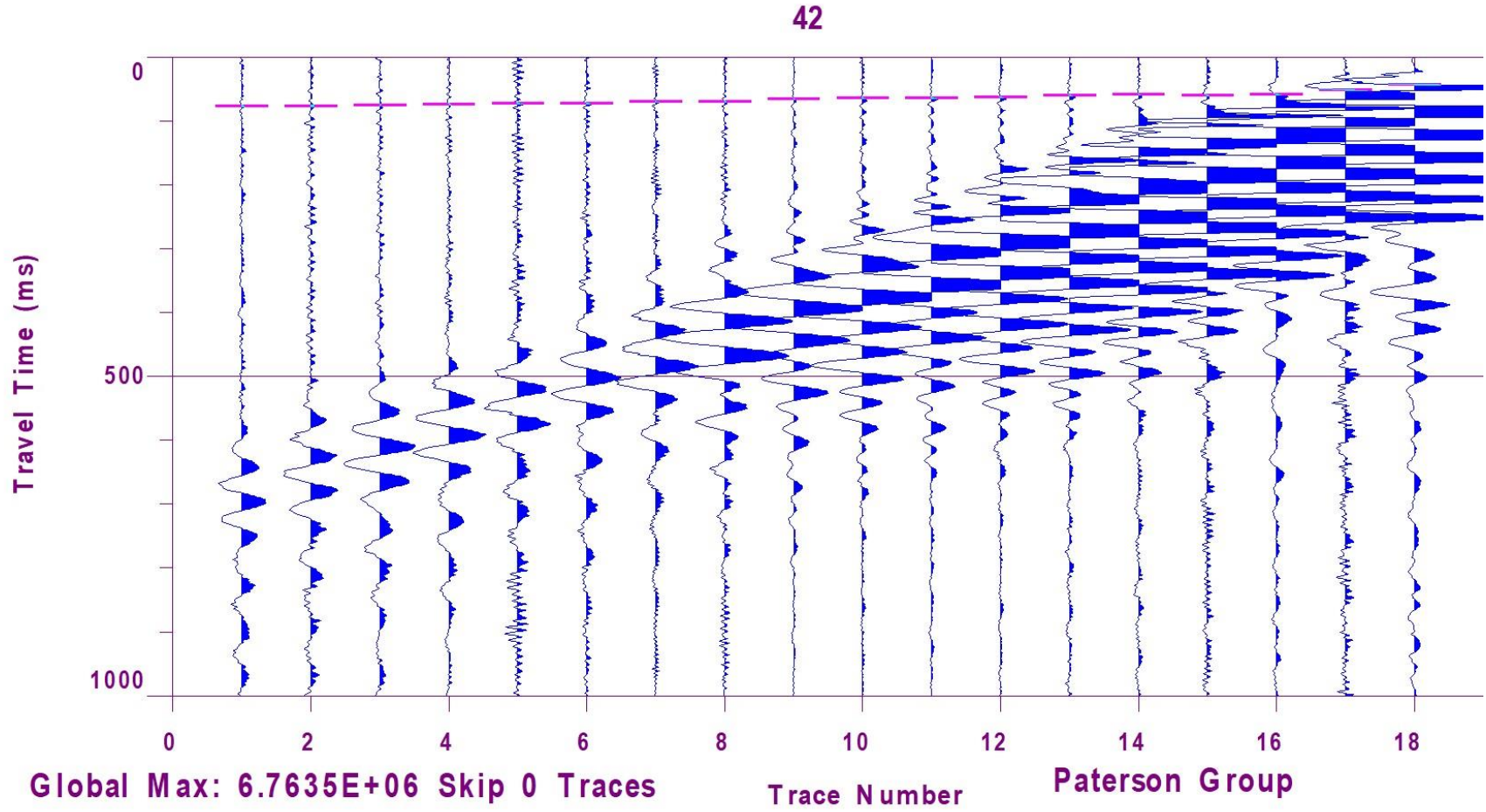**DRAWING PG5321-1 - TEST HOLE LOCATION PLAN**

FIGURE 1

KEY PLAN

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Figure 2 – Shear Wave Velocity Profile at Shot Location +55.5 m
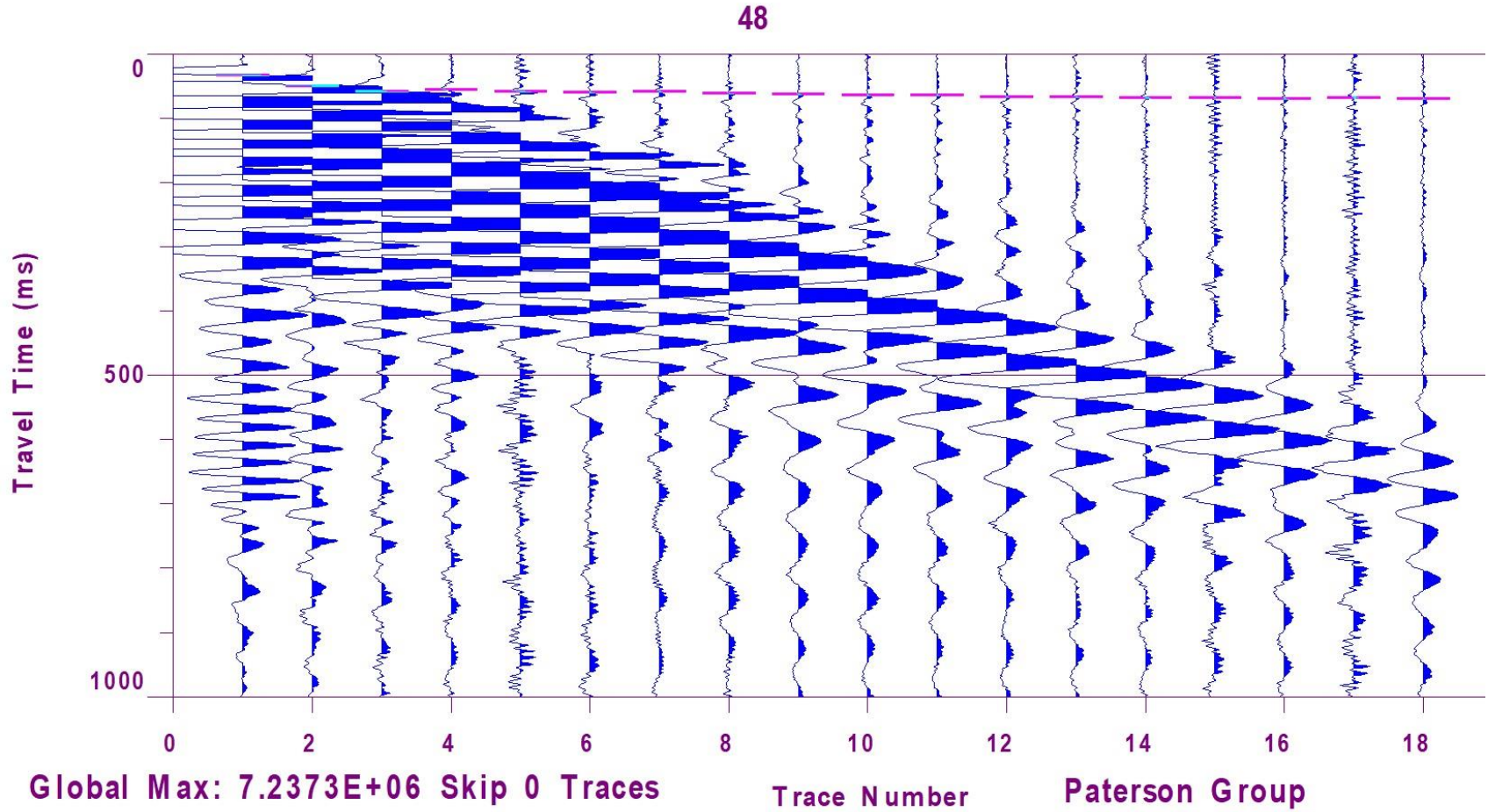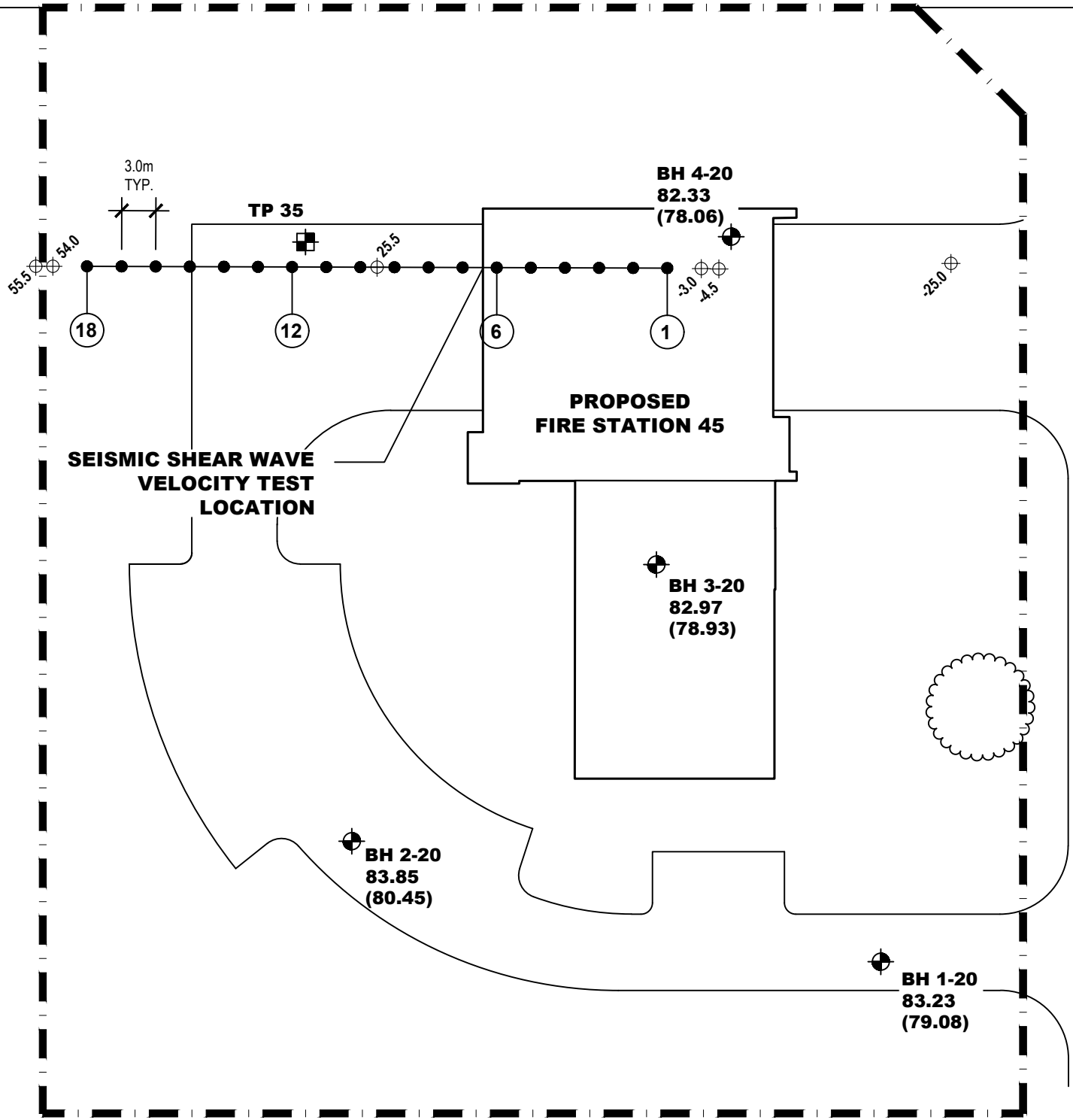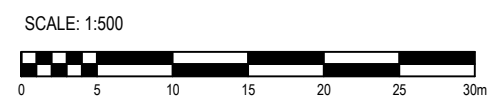
patersongroup

Figure 3 – Shear Wave Velocity Profile at Shot Location -3.0 m

M A R C H R O A D

3.0m
TYP.

TP 35

BH 4-20
82.33
(78.06)

76.0

55.5 54.0

25.5

-3.0 -4.5

-25.0

18

12

6

1

PROPOSED
FIRE STATION 45

SEISMIC SHEAR WAVE
VELOCITY TEST
LOCATION

BH 3-20
82.97
(78.93)

BH 2-20
83.85
(80.45)

BH 1-20
83.23
(79.08)

-3.0
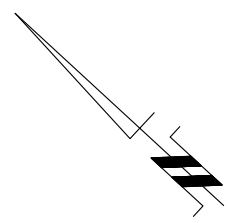
LEGEND:

⊕ BOREHOLE LOCATION

◆ TEST PIT LOCATION, PATERSON GROUP
    REPORT PG2878, 2013

82.94 GROUND SURFACE ELEVATION (m)

(78.79) PRACTICAL REFUSAL TO AUGERING
         ELEVATION (m)

●—● GEOPHONE LOCATIONS

18 GEOPHONE NUMBER

⊕ -3.0 SHOT LOCATION

SCALE: 1:500

0 5 10 15 20 25 30m

NO.	REVISIONS	DATE	INITIAL

CITY OF OTTAWA
GEOTECHNICAL INVESTIGATION
PROPOSED FIRE HALL - 1075 MARCH ROAD

OTTAWA,   ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale:	1:500	Date:	05/2020
Drawn by:	MPG	Report No.:	PG5321-1
Checked by:	JV	Dwg. No.:	**PG5321-1**
Approved by:	DJG	Revision No.:	

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11x17