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Archaeological Services

Geotechnical Investigation

Proposed Residential Development
3817 - 3843 Innes Road
Ottawa, Ontario

Prepared For

Oligo Group

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Report: PG5275-1 Revision 1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Oligo Group to prepare a geotechnical investigation report update for the proposed residential development to be located at 3817 - 3843 Innes Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

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

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

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

2.0 Proposed Development

The proposed development is understood to consist of several 4-storey buildings with a separate underground parking level per building. Associated at-grade parking, access lanes, landscaped areas are also anticipated as part of the development. The site is anticipated to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on May 13 and 14, 2009. At that time, five (5) boreholes were advanced to a maximum depth of 5.1 m. The borehole locations were distributed in a manner to provide general coverage of the subject site. The locations of the boreholes are shown on Drawing PG5275-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

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

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

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

Groundwater

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

3.2 Field Survey

The borehole locations were selected, determined in the field and surveyed by Paterson. Ground surface elevations at the test hole locations were referenced to a temporary benchmark (TBM), consisting of the top spindle of the fire hydrant located across from 3843 Innes Road. A geodetic elevation of 92.46 m was provided for the TBM by A. Dagenais and Associates Inc. The locations of the boreholes are shown on Drawing PG5275-1 - Test Hole Location Plan attached to this report.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the field logs.

3.4 Analytical Testing

One (1) 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 soil. The analytical test results are presented in Appendix 1 and discussed in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

At the time of the investigation, the subject site was occupied by several residential dwellings with the majority of the site being grass covered with occasional mature trees and gravel covered driveways. The site is relatively flat, sloping gradually downward toward the southeast and is slightly above the grade of Innes Road.

The site is bordered to the east by a gas station, to the south by Innes Road and to the north and west by residential properties.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of a topsoil layer underlain by compact silty sand fill, a native compact sandy silt and/or compact to dense glacial till. Practical refusal to augering was encountered at each test hole location on an inferred bedrock surface at depths varying between 1.47 and 5.1 m. Specific details of the soil profile at each borehole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and dolomite of the Gull River Formation, which is expected to be encountered at a 1 to 3 m depth.

4.3 Groundwater

Groundwater levels were measured in the standpipes on May 19, 2009. All piezometers were noted to be dry, except BH 1, where a reading of 2.74 m was observed. The long-term groundwater table is estimated to be between 3 to 4 m depth based on the colour and consistency of the recovered soil samples at the test hole locations. Groundwater levels are subject to seasonal fluctuations and could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed development. The proposed structures are expected be founded by conventional shallow footings placed on glacial till and/or bedrock bearing surface or over zero entry, vertical concrete in-filled trenches placed over glacial till or bedrock bearing surface.

Depending on the depth of the proposed footings, bedrock removal may be required in areas across the site for building construction and service installation. Bedrock removal recommendations are summarized in the following section.

Due to the undulating bedrock surface anticipated at the founding level, engineered fill or unshrinkable fill (20 MPa Lean Concrete) can be used to provide a level surface for the placement of the footings.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious 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 proposed building perimeter. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.

The existing fill layer should be reviewed by the geotechnical consultant, once a large area of the fill is exposed to determine if the existing fill will provide an acceptable subgrade surface on which to commence backfilling for the proposed pavement structure.

Fill Placement

Fill used for grading beneath the proposed building, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building area should be compacted to at least 98% of 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 at least compacted by the tracks of the spreading equipment to minimize voids. If site excavated material, free of organics and deleterious materials, is to be used to build up the subgrade level for areas to be paved, the site excavated silty sand, under dry conditions, 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.

If excavated bedrock is to be placed as backfill, the bedrock should be crushed to produce a well-graded material, similar to an OPSS Granular B Type II material and approved by the geotechnical consultant. Where the crushed bedrock is open-graded, a binding layer with a woven geotextile, such as Terratrack 200 or equivalent, is recommended to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be determined at the time of construction.

Lean Concrete Filled Trenches

As an alternative to placing engineered fill, where required, consideration should be given to excavating vertical trenches to the undisturbed, compact to dense glacial till or clean, surface sounded bedrock, and backfilling 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 glacial till or bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

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.

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 50 mm per second during the blasting program to reduce the risks of damage to the existing structures.

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 or to provide a stable base for the overburden shoring system.

Vibration Considerations

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

The following construction equipments could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether it is caused by blasting operations or by construction operations could be the cause of the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

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

5.3 Foundation Design

Soil Bearing Medium

Footings placed on an undisturbed, compact sandy silt or compact glacial till or engineered fill placed on the above noted bearing surface can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the above-noted bearing resistance value at ULS.

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.

Footings designed using the above-noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

Bedrock Bearing Medium

Footings placed on a clean, surface sounded bedrock or concrete in-filled trenches extended to a bedrock bearing surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **500 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS.

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

The potential long term post-construction total and differential settlements for footings placed on surface-sounded bedrock are estimated to be negligible.

Soil/Rock Transition

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long term total and differential settlements. Also, at the soil/bedrock and bedrock/soil transitions, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material. The width of the subexcavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a compact sandy silt or glacial till above the groundwater table when a plane extending horizontally and vertically from the footing perimeter a minimum of 1.5H:1V passing through in situ soil of the same or higher capacity as the bearing medium soil.

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 sound bedrock bearing media. A 1H:1V slope can be used for fractured/weathered bedrock.

5.4 Design for Earthquakes

A shear wave velocity test was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building from Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity test was completed by Paterson personnel. The shear wave velocity test results were from two shot locations are presented in Figures 2 and 3 in Appendix 2.

Field Program

The shear wave test location is presented in Drawing PG5275-1 - Test Hole Location Plan. Paterson field personnel placed 24 horizontal geophones in a straight line in an approximate east-west orientation. The 4.5 Hz. horizontal geophones were mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 3 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph. The seismograph was also connected to a computer laptop and a trigger switch attached to a 12 pound dead blow hammer. The 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 shot is repeated between 5 to 10 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 20 m away from the first and last geophone.

The test method completed by Paterson is 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 of the results were completed by Paterson personnel. The shear wave velocity measurement was calculated 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, V_{s30}, of the upper 30 m profile, immediately below the proposed foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location. The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. As bedrock quality increases, generally the bedrock shear wave velocity increases.

The proposed building is expected to be founded on shallow footings. Based on the field investigation, bedrock depths vary between 0.9 to 3.6 m depth across the subject site. Based on our analysis, the average bedrock and overburden soil shear wave velocity was calculated to be 2,458 m/s and 309 m/s, respectively.

The V_{s30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012, as presented on the following page for footings founded directly over the bedrock surface.

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

$$V_{s30} = \frac{30m}{\sum\left(\frac{3.0m}{309m/s} + \frac{27m}{2{,}458m/s}\right)}$$

$$V_{s30} = 1449m/s$$

Based on the seismic test results, the average shear wave velocity, Vs30, for foundations placed on overburden within 3 m above the bedrock surface, is 1,449 m/s and **Site Class B** is applicable as per Table 4.1.8.4.A of the OBC 2012. A seismic **site Class A** is applicable for footings placed directly on bedrock or over concrete in-filled trenches extended to the bedrock surface. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil, and deleterious fill, containing organic matter, within the footprint of the proposed building, the glacial till or bedrock surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

The upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to 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, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

The majority of the foundation walls are expected be above the long term groundwater level; therefore, the retained soils should be considered drained. However, if undrained conditions are anticipated (i.e. below the groundwater level), 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 for all subsurface units below the watertable when calculating the effective unit weight. The total earth pressure (P_{AE}) includes both the static earth pressure component (P_A) and the seismic component (ΔP_{AE}).

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. Note that 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 stay at least 0.3 m away from the walls with the compaction equipment.

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. Note that the vertical seismic coefficient is assumed to be zero.

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

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

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

5.7 Pavement Structure

Recommended Pavement Structure

Car only parking areas and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 1 and 2.

Table 1 - 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 2 - 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 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 vibratory equipment.

6.0 Design and Construction precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended to 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 19 mm clear crushed stone surrounded by geotextile and 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. A composite foundation drainage board, such as Delta Drain 6000, is also recommended to be installed over the vertical foundation walls and connect to the perimeter drainage pipe.

Backfill

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

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover in conjunction with foundation insulation, should be provided in this regard.

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

Frost Susceptibility of Bedrock

If bedrock is encountered above the frost depth and soil frost cover is less than the 1.5 m, the frost susceptibility of the bedrock should be determined. This can be accomplished as follows:

- ❏ Drill probeholes within the bedrock and assess the frost susceptibility.

- ❏ Examine service trench profiles extending in bedrock in the vicinity of the foundation to determine if weathering is extensive.

If the bedrock is considered to be **non-frost susceptible**, the footings can be poured directly on the bedrock without any further frost protective measures.

If the bedrock is considered to be **frost susceptible**, the following measures should be implemented for frost protection:

- ❏ Option A - Sub-excavate the weathered bedrock to sound bedrock or to the required frost cover depth. Pour footings at the lower level.

- ❏ Option B - Provide insulation to protect footings. Pouring footings on the insulation overlying weathered bedrock is preferred. However, due to potential undulating bedrock surface, consideration may have to be given to adopting an insulation detail that allows the footing to be poured directly on the weathered bedrock.

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.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

6.4 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes.

Cover material, from the spring line to at least 300 mm above the obvert of the pipe 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% of the material’s 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 the potential 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

It is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. 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) Category 3 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, 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.

Impacts on Neighbouring Structures

Based on our observations, the groundwater within the subject site is expected at a depth of 3 to 4 m below existing grade which is within the bedrock. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site. Based on the proximity of the neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures. It should be noted that no issues are expected with respect to the groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed buildings.

6.6 Winter Construction

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

The subsoil conditions at this site mostly consist of frost susceptible materials. In 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.

Precaution must be taken where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.7 Corrosion Potential and Sulphate

The analytical test results indicate that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement would be appropriate. The results of the chloride content, pH and resistivity indicate the presence of a slightly aggressive environment for exposed ferrous metals.

7.0 Recommendations

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

- ❏ Observation of all bearing surfaces prior to the placement of concrete.
- ❏ Sampling and testing of the concrete and fill materials used.
- ❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❏ Observation of all subgrades prior to backfilling.
- ❏ Field density tests to determine the level of compaction achieved.
- ❏ Sampling and testing of the bituminous concrete including mix design reviews.

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

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well as the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification 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 Oligo Group or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Faisal I. Abou-Seido, P.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- ❑ Oligo Group (3 copies)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

PARACEL LABORATORIES LTD.

Order #: 0921077

Certificate of Analysis

Report Date: 26-May-2009
Order Date:14-May-2009

Client: **Paterson Group Consulting Engineers**
Client PO: 4210 Project Description: PG1854

	Client ID:	BH1 SS3	-	-	-
	Sample Date:	13-May-09	-	-	-
	Sample ID:	0921077-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	90.5	-	-	-
General Inorganics					
pH	0.05 pH Units	8.42	-	-	-
Resistivity	0.10 Ohm.m	85.2	-	-	-
Anions					
Chloride	5 ug/g dry	11	-	-	-
Sulphate	5 ug/g dry	9	-	-	-

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Condominium Development - 3843 Innes Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, across 3835 Innes Road. Geodetic elevation = 92.46m.

REMARKS

BORINGS BY CME 850 Power Auger

DATE May 13, 2009

FILE NO. PG1854

HOLE NO. BH 1

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	92.03		
TOPSOIL 0.05									
FILL: Brown sandy silt with clay and loam 0.69		AU	1						
Loose, brown **SANDY SILT**, some clay 1.45		SS	2	42	8	1	91.03		
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and shale fragments		SS	3	79	23	2	90.03		
		SS	4	42	42	3	89.03		
		SS	5	50	51	4	88.03		
		SS	6	79	25	5	87.03		
5.18		SS	7	50	66				

End of Borehole

Practical refusal to augering @ 5.18m depth

(GWL @ 2.74m-May 19/09)

Pen. Resist. / Water Content % scale: 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
Prop. Condominium Development - 3843 Innes Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, across 3835 Innes Road. Geodetic elevation = 92.46m.

REMARKS

BORINGS BY CME 850 Power Auger

DATE May 14, 2009

FILE NO. PG1854

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	93.95		
TOPSOIL 0.08									
FILL: Brown sandy silt with clay, gravel and brick		AU	1						
		SS	2	100	50+	1	92.95		
1.47		AU	3						
End of Borehole Practical refusal to augering @ 1.47m depth (BH dry upon completion)									

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
Prop. Condominium Development - 3843 Innes Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, across 3835 Innes Road. Geodetic elevation = 92.46m.

REMARKS

BORINGS BY CME 850 Power Auger **DATE** May 14, 2009

FILE NO. PG1854

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	92.49
TOPSOIL — 0.05							
FILL: Brown silty clay with cobbles and rock fragments — 0.84		AU	1				
		SS	2	38	14	1	91.49
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders		SS	3	71	59	2	90.49
— 2.44		SS	4	60	50+		
End of Borehole							
Practical refusal to augering @ 2.44m depth							
(BH dry-May 19/09)							

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Condominium Development - 3843 Innes Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, across 3835 Innes Road. Geodetic elevation = 92.46m.

REMARKS

BORINGS BY CME 850 Power Auger

DATE May 14, 2009

FILE NO. PG1854

HOLE NO. BH 4

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 % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	91.73		
13mm Topsoil over brown silty clay with sandy loam **FILL** (0.69)		AU	1						
Loose, brown **SANDY SILT** with clay (1.45)		SS	2	67	8	1	90.73		
GLACIAL TILL: Compact, brown sandy silt with clay, gravel, cobbles and shale fragments		SS	3	42	11	2	89.73		
(2.59)		SS	4	57	50+				
Weathered **BEDROCK** (2.74)									
End of Borehole Practical refusal to augering @ 2.74m depth (BH dry-May 19/09)									

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
Prop. Condominium Development - 3843 Innes Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, across 3835 Innes Road. Geodetic elevation = 92.46m.

REMARKS

BORINGS BY CME 850 Power Auger

DATE May 14, 2009

FILE NO. PG1854

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.02
FILL: Black topoil with silty clay, trace gravel		AU	1				
0.69							
GLACIAL TILL: Brown sandy silt with clay, gravel, cobbles and boulders		SS	2	75	15	1	92.02
		SS	3	67	36	2	91.02
		SS	4	80	50+		
2.95							
End of Borehole Practical refusal to augering @ 2.95m depth (BH dry-May 19/09)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

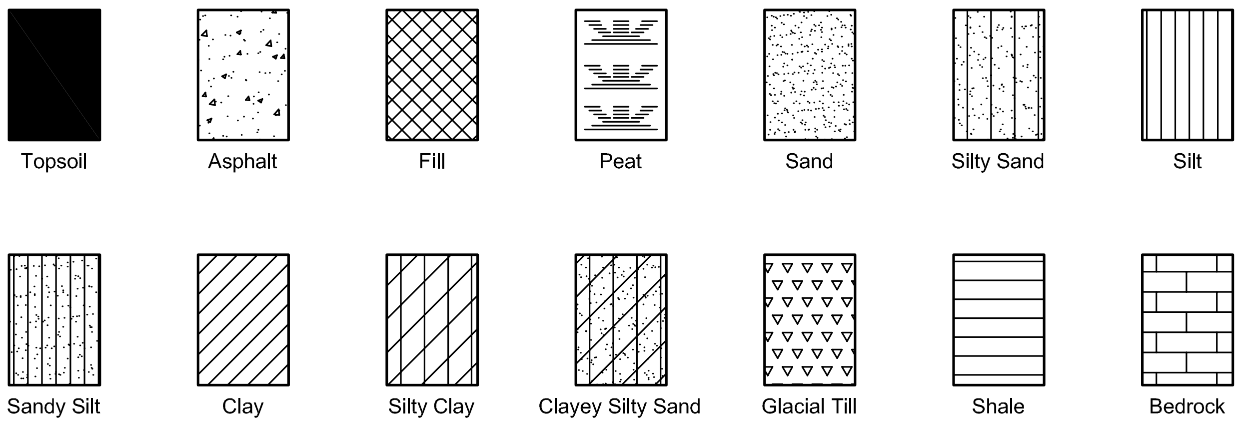
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

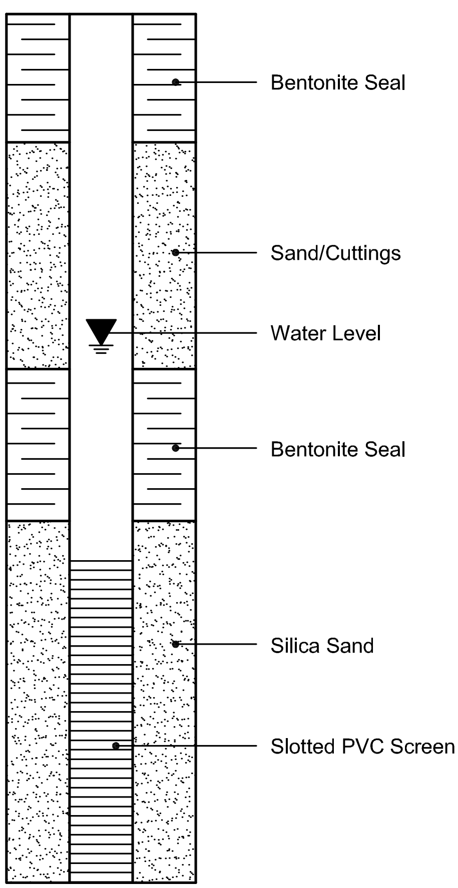
SYMBOLS AND TERMS (continued)

STRATA PLOT

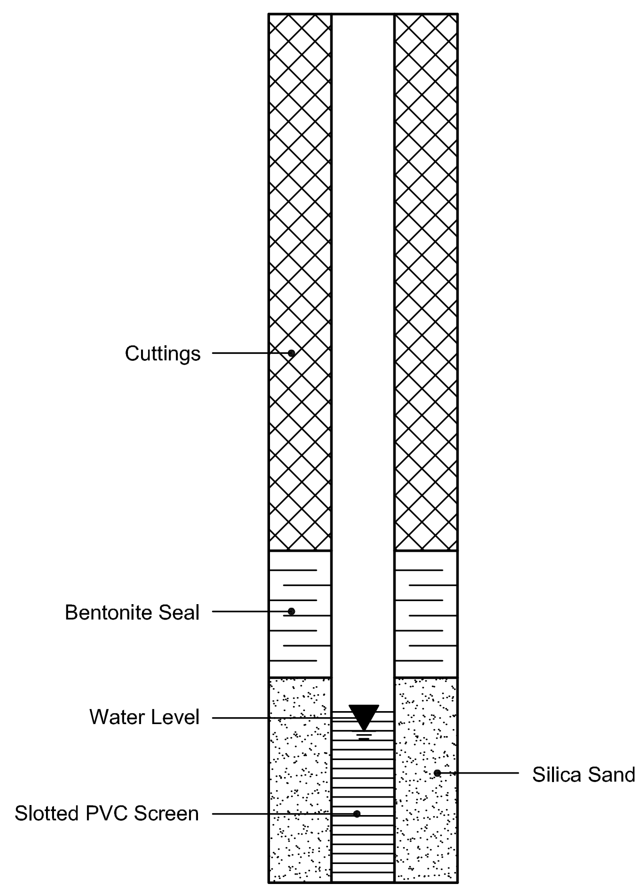


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SEISMIC SHEAR WAVE VELOCITIES

DRAWING PG5275-1 - TEST HOLE LOCATION PLAN

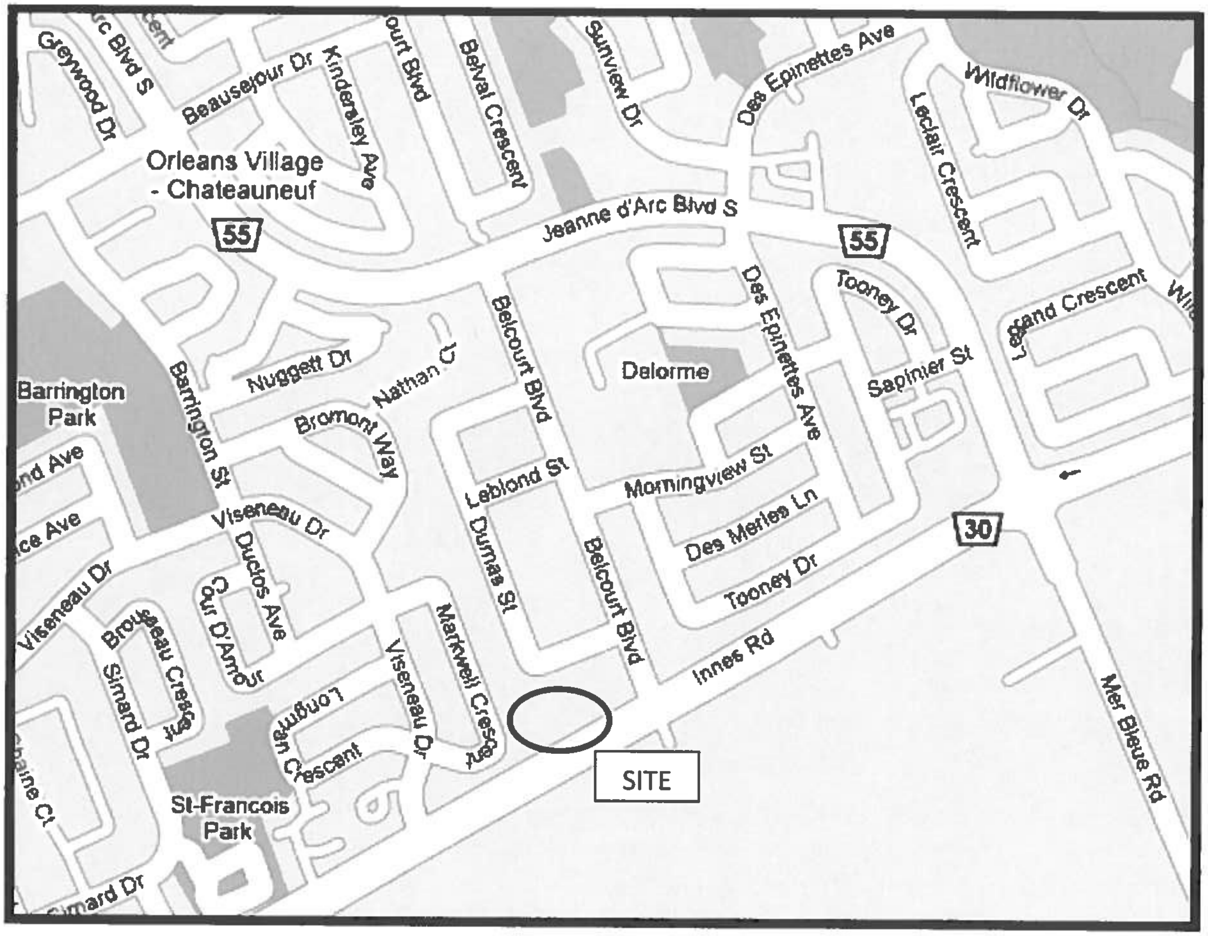


FIGURE 1

KEY PLAN

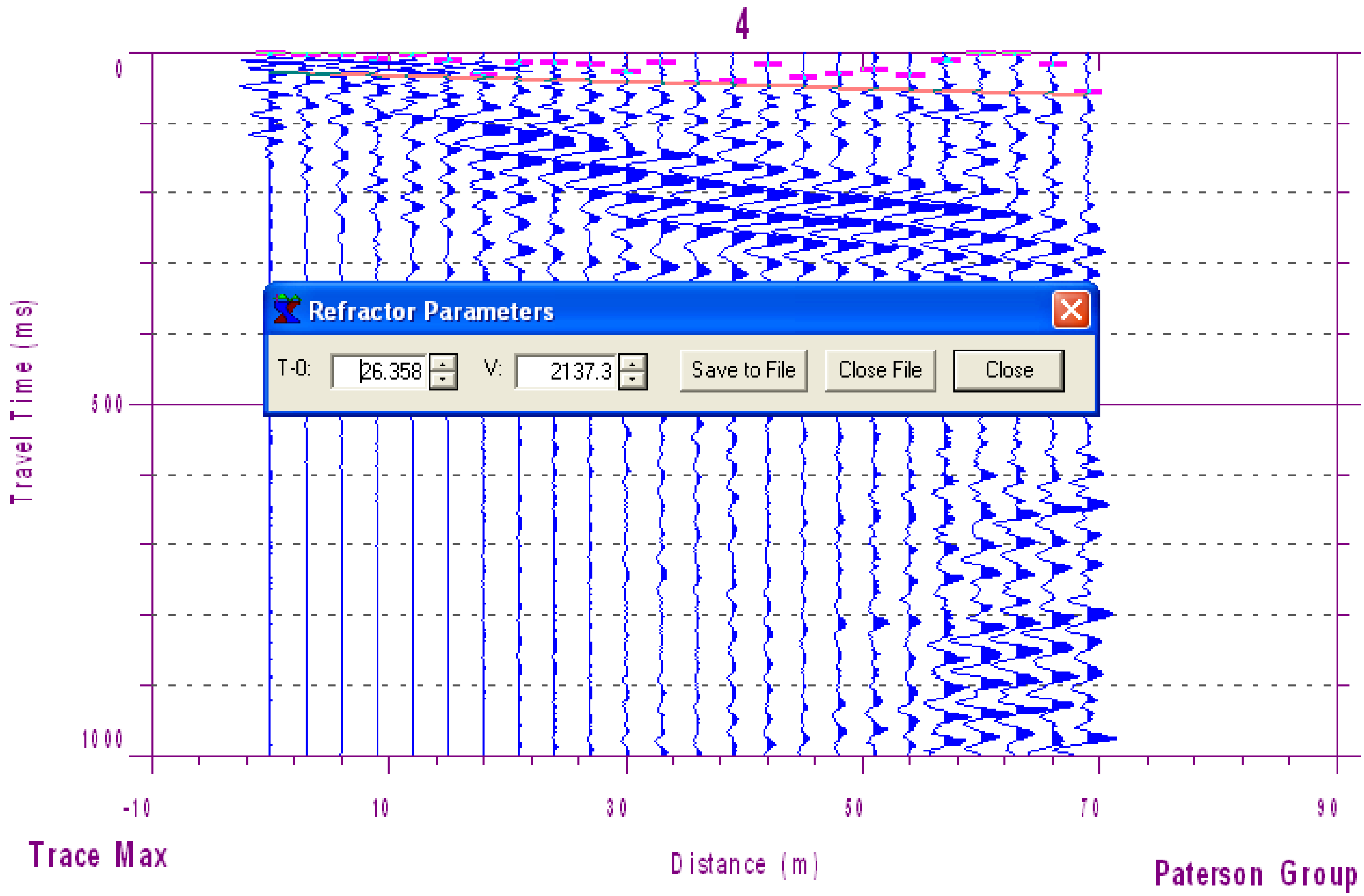


Figure 2 – Shear Wave Velocity Profile at Shot Location -4.5

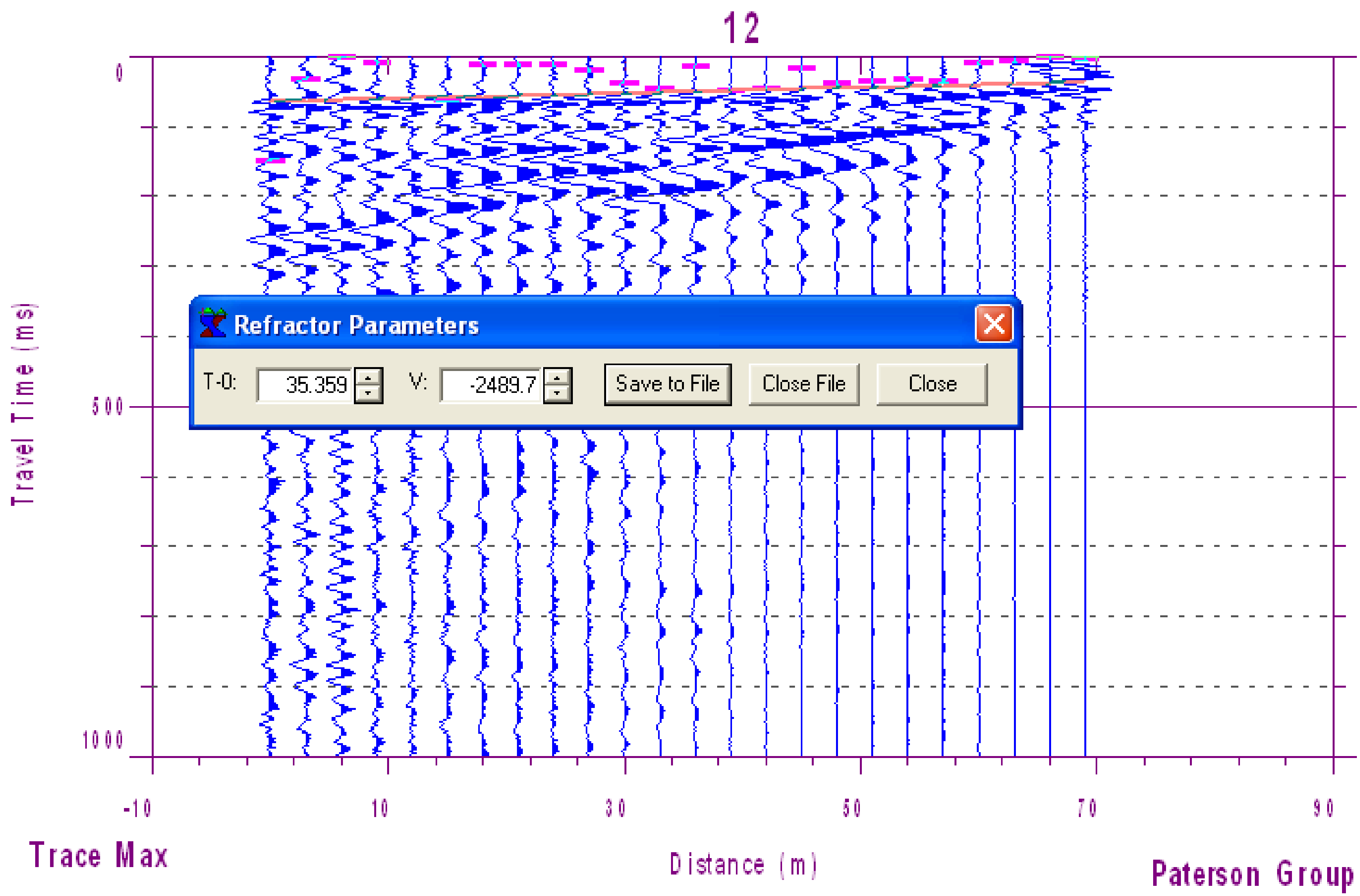
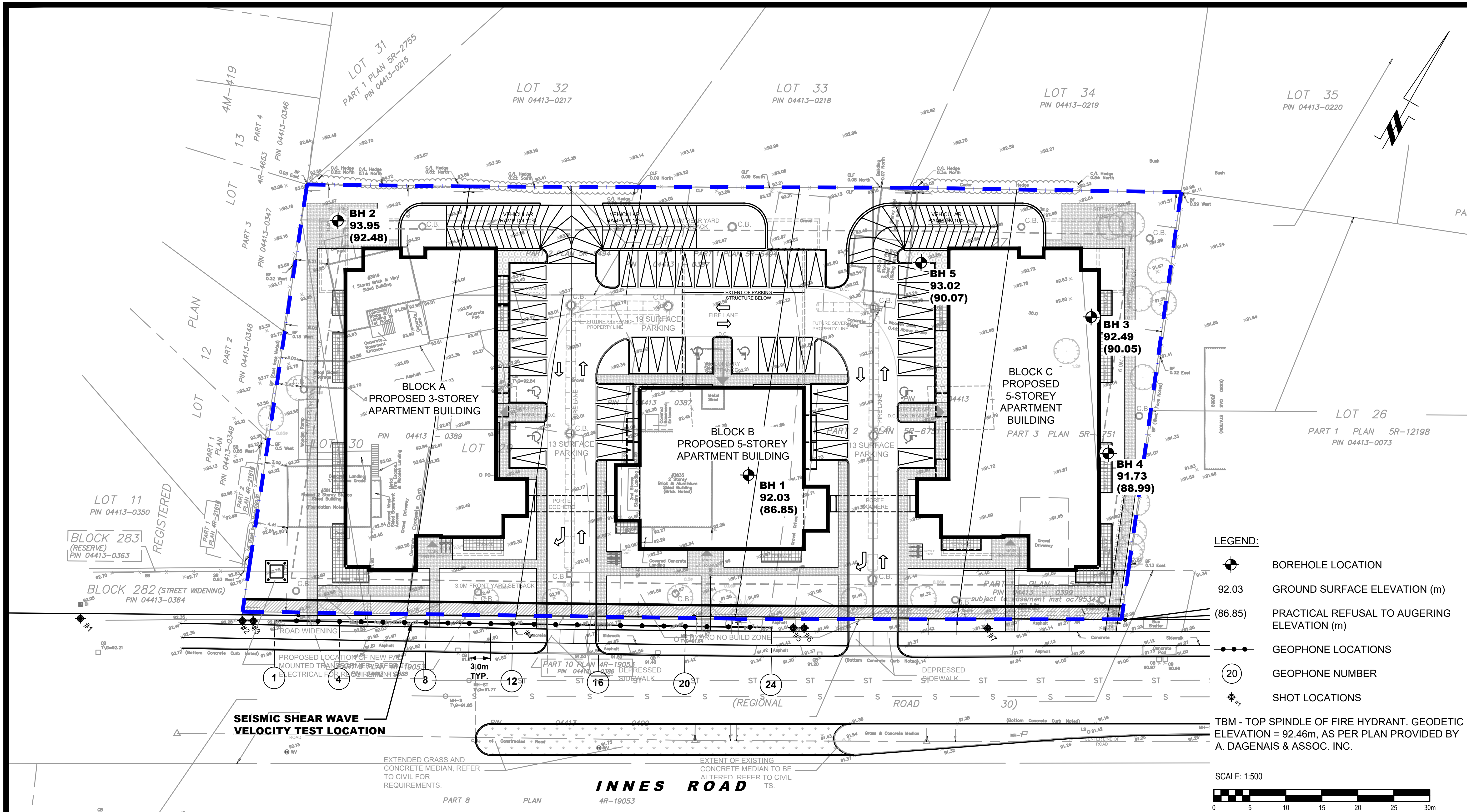


Figure 3 – Shear Wave Velocity Profile at Shot Location 73.5



p:\autocad drawings\geotechnical\pg52xx\pg5275\pg5275-1 test hole location plan (rev.01).dwg

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NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO NEW CONCEPTUAL PLAN	30/03/2021	FA

OLIGO GROUP

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT - 3817-3843 INNES ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 02/2020
Drawn by: NFRV	Report No.: PG5275-1
Checked by: CB	Dwg. No.: PG5275-1
Approved by: DJG	Revision No.: 1