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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

Geotechnical Investigation

Proposed Commercial Building
3636 Innes Road
Ottawa, Ontario

Prepared For

Amerco Real Estate Company

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Report: PG5292-1 Revision 1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Amerco Real Estate Company to conduct a geotechnical investigation for the proposed commercial building to be located at 3636 Innes Road, in the City of Ottawa, Ontario (Refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current geotechnical review were:

- ❏ to determine the subsurface soil and groundwater conditions based on the test holes.
- ❏ to provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a three-storey, commercial building of slab-on-grade construction. Associated car parking and access lanes, loading docks and landscaped areas are also anticipated. It is further anticipated that the site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current investigation was conducted on March 26, 2020 and consisted of advancing four (4) boreholes to a maximum depth of 3.2 m below the existing ground surface. The borehole locations were distributed in a manner to provide general coverage of the subject site. The locations of the test holes are shown on Drawing PG5292-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed 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 from our geotechnical division under the direction of a senior engineer. The test hole procedures consisted of advancing the boreholes to the required depths at the selected locations and sampling 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 locations and ground surface elevations at each test hole location were surveyed by Paterson personnel and referenced to a geodetic datum using a Trimble GPS unit. The test hole locations and ground surface elevations at the test hole locations are presented on Drawing PG5292-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were visually examined in our laboratory to review the results of the field logging. Grain size distribution (hydrometer) testing was completed on two (2) soil samples. The results are presented in Subsection 4.2 and Appendix 1.

3.4 Analytical Testing

One (1) soil sample was submitted to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are discussed in Subsection 6.7 and shown in Appendix 1.

4.0 Observations

4.1 Surface Conditions

Generally, the ground surface across the subject site is relatively flat and consists of paved and gravel surfaced areas. The north, west and east portions of the subject site are currently occupied by one to two-storey commercial storage buildings. The site is currently bordered to the north by Innes Road and to the east, south and west by vacant undeveloped land.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consisted of an asphalt surfaced pavement structure underlain by a layer of silty clay fill to depths between 0.6 to 1.1 m. The fill layer was observed to be underlain by a native deposit of very stiff to stiff brown silty clay, which is further underlain by a deposit of glacial till. Where encountered, the fine matrix of the glacial till was observed to consist of silty clay with sand, gravel, cobbles and boulders. Practical refusal to augering was encountered at all boreholes at depths between 2.6 m to 3.2 m. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Hydrometer Testing

Two (2) soil samples were submitted for hydrometer testing. The results are summarized in Table 1 and presented on the Grain Size Distribution sheets in Appendix 1.

Table 1 - Summary of Hydrometer Tests

Sample	Gravel %	Sand %	Fines Content	
			Silt %	Clay %
BH2 SS4	0	8.1	25.4	68.5
BH4 SS4	8.4	13.4	33.7	46.5

Bedrock

Based on available geological mapping, the local bedrock consists of limestone of the Bobcaygeon formation with an anticipated overburden thickness of 2 to 4 m.

4.3 Groundwater

The groundwater levels were measured in the borehole locations on April 2, 2020, and are presented in the Soil Profile and Test Data sheets in Appendix 1.

Table 2 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH1	90.07	0.18	89.89	April 2, 2020
BH2	90.22	Surface	90.22	April 2, 2020
BH3	90.08	0.44	89.64	April 2, 2020
BH4	89.94	0.50	89.44	April 2, 2020

Note: The ground surface elevations were surveyed by Paterson personnel and referenced to a geodetic datum.

It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled borehole, which can lead to higher than typical groundwater levels. Long-term groundwater levels can also be determined based on field observations, such as soil colouring, moisture levels and consistency. Based on these observations at the borehole locations, the long-term groundwater level is expected at 2 to 3 m depth. It should be noted that groundwater levels fluctuate periodically throughout the year and higher levels could encountered at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered adequate for the proposed development. It is expected that the proposed building can be founded on conventional style shallow foundations placed on an undisturbed, stiff silty clay or compact glacial till bearing surface.

Depending on the depth of site servicing, the bedrock may be encountered during excavation. Additionally, due to the presence of a silty clay deposit underlying the subject site, a permissible grade raise restriction will be required.

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

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, paved areas, pipe bedding and other settlement sensitive structures. It is anticipated that the existing fill, free of significant amounts of deleterious material and organics, can be left in place below the proposed building footprint outside of lateral support zones for the footings. However, it is recommended that the slab subgrade surface be proof-rolled **under dry conditions and above freezing temperatures** by an adequately sized roller making several passes to achieve optimum compaction levels. The compaction program should be reviewed and approved by Paterson at the time of construction.

In poor performing areas, the existing fill should be removed and replaced with an approved engineered fill. Care should be taken not to disturb adequate bearing soils below the subgrade level during site preparation activities.

Bedrock Removal

Due to the close proximity of the bedrock surface, bedrock removal may be required to complete site servicing activities. Bedrock removal can be accomplished by hoe ramming where only small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or hoe ramming.

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

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

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

Excavation side slopes in sound bedrock can be excavated almost vertical side walls. A minimum 1 m horizontal ledge, should remain between the overburden excavation and the bedrock surface. The ledge will provide an area to allow for potential sloughing or a stable base for the overburden shoring system.

Vibration Considerations

Construction operations are 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 caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to 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). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Fill Placement

Fill used for grading beneath the building areas 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 building 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 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.

5.3 Foundation Design

Bearing Resistance Values

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, founded on an undisturbed, silty clay or glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and at ultimate limit states (ULS) of **300 kPa**. A geotechnical resistance factor of 0.5 was incorporated into the above-noted bearing resistance values at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed, in the dry, prior to the placement of concrete for footings.

Footings bearing on an undisturbed soil bearing surface and designed using the bearing resistance values provided herein will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

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 an undisturbed soil bearing surface above the groundwater table when a plane extending horizontally and vertically from the bottom edge of the footing at a minimum of 1.5H:1V passes through in situ soil of the same or higher capacity as the bearing medium soil.

Permissible Grade Raise

Based on experience with the local silty clay deposit, a permissible grade raise restriction for the subject site of **2.0 m** can be used for design purposes.

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 PG5292-1 - Test Hole Location Plan presented in Appendix 2. Paterson field personnel placed 18 horizontal geophones in a straight line in roughly a north-south orientation. The 4.5 Hz. horizontal geophones were mounted to the surface by means of a 75 mm ground spike attached to the geophone land case. The geophones were spaced at 3 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between 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 **245 m/s**, while the bedrock shear wave velocity is **2,974 m/s**. It is anticipated that the underside of footing will be located at an approximate geodetic elevation of 89.3 m. Based on the anticipated design USF elevation, up to 2.5 m of overburden may be present between the underside of the footing and bedrock. The Vs_{30} was calculated using the standard equation for average shear wave velocity provided in OBC 2012, and as presented below:

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

$$V_{s30} = \frac{30m}{\left(\frac{2.5m}{245m/s} + \frac{27.5m}{2{,}974m/s}\right)}$$

$$V_{s30} = 1{,}542m/s$$

Based on the results of the seismic shear wave velocity testing, the average shear wave velocity, Vs_{30}, for the proposed building is **1,542 m/s** for an anticipated underside of footing at approximate geodetic elevation of 89.3 m. Therefore, for the anticipated underside of footing elevation noted above, 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

With the removal of all topsoil and fill, containing deleterious or organic materials, the native soil or existing fill approved by the geotechnical consultant at the time of construction will be considered to be an acceptable subgrade surface on which to commence backfilling for slab on grade construction.

It is recommended that the existing fill be proof-rolled with a suitably sized roller making several passes under dry conditions and above freezing temperatures prior to fill placement and approved by the geotechnical consultant. Any poor performing areas should be removed and backfilled with appropriate backfill material. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

It is recommended that the upper 300 mm of sub-floor fill should consist of OPSS Granular A crushed stone. All backfill materials within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

5.6 Pavement Structure

Car only parking areas, access lanes and heavy truck parking areas are anticipated at this site. The proposed pavement structures are shown in Tables 3 and 4.

Table 3 - 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 in situ soils or OPSS Granular B Type I or II material placed over in situ soil.	

Table 4 - Recommended Pavement Structure - Access Lanes and Heavy Truck Loading/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 in situ soils or OPSS Granular B Type I or II material placed over in situ soil	

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

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick loose 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 keeping the contact zone between the subgrade material and the base stone in a dry condition.

Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing the load bearing capacity.

Where silty clay is anticipated at subgrade level, consideration should be given to installing sub-drains at the catch basin locations during the pavement construction. The sub-drain inverts should be approximately 300 mm below subgrade level and extend 3 m along the curblines in both directions. 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

It is recommended that a perimeter foundation drainage system be provided for the proposed building. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone 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.

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

Concrete Sidewalks Adjacent to Building

To avoid differential settlements within the proposed sidewalks adjacent to the proposed building, it is recommended that the upper 600 mm of backfill placed below the concrete sidewalks adjacent to the building footprint to consist of non-frost susceptible material such as OPSS Granular A or Granular B Type II. The granular material should be placed in maximum 300 mm loose lifts and compacted to 98% of the material's SPMDD using suitable compaction equipment. The subgrade material should be shaped to promote positive drainage towards the building's perimeter drainage pipe. Consideration should be given to placing a rigid insulation layer below the granular fill layer to prevent frost heave issues at the building entrances.

6.2 Protection Against Frost Action

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

Exterior footings, such as those for isolated exterior piers and loading docks, are more prone to deleterious movement associated with frost action than the exterior walls of heated structures and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

It is recommended that the geotechnical consultant review the proposed frost protection detail for any loading dock footings.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

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

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

6.4 Pipe Bedding and Backfill

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

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. The bedding thickness should be increased to 300 mm for areas where the subgrade consists of bedrock. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum particle size of 25 mm.

The material should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of its 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 minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD.

It should generally be possible to re-use the moist (not wet) silty clay and glacial till above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet site-excavated materials will be difficult to re-use, as the high water contents make compacting impractical without an extensive drying period. Furthermore, all cobbles greater than 200 mm in the longest dimension should be removed prior to the site materials being reused.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium. It is anticipated that groundwater infiltration into the excavations should be low and controllable using conventional open sumps.

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, 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

The subsurface 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. Precautions should 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 use of straw, propane heaters, 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.

The trench excavations should be constructed in a manner that will avoid the introduction of frozen materials into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. In addition, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure. 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 results of the chloride content, pH and resistively indicate the presence of a moderate to aggressive environment for exposed ferrous metals at this site.

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:

- ❏ Review of the final grading plan from a geotechnical perspective, once available.
- ❏ 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 made 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 is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test hole locations, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on our undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Amerco Real Estate Company or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
June 30-2020
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B.Eng.

David J. Gilbert, P.Eng

Report Distribution:

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- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

GRAIN SIZE DISTRIBUTION TESTING RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Invenstigation
Proposed 3-Storey Building - 3636 Innes Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5292**

REMARKS

HOLE NO. **BH 1**

BORINGS BY CME-55 Low Clearance Drill **DATE** March 26, 2020

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	90.07
Asphaltic concrete	0.05						
FILL: Compact, brown silty sand with clay and gravel	1.07	SS	1	50	15		
		SS	2	4	13	1	89.07
Very stiff, brown **SILTY CLAY**	2.29	SS	3	88	10	2	88.07
GLACIAL TILL: Loose to very dense, brown silty clay with sand, gravel, cobbles and boulders	3.20	SS	4	33	6		
		SS	5		50+	3	87.07

End of Borehole

Practical refusal to augering at 3.20m depth

(GWL @ 0.18m - April 2, 2020)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Invenstigation
Proposed 3-Storey Building - 3636 Innes Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE March 26, 2020

FILE NO. PG5292

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	90.22
Asphaltic concrete (to 0.10)							
FILL: Brown silty sand with crushed stone (0.10 to 0.59)		SS	1	46	34		
FILL: Compact, brown silty clay with sand and gravel (0.59 to 1.01)						1	89.22
Very stiff to stiff, brown **SILTY CLAY** (1.01 to 2.84)		SS	2	17	10		
		SS	3	88	10	2	88.22
- with sand and gravel and grey by 2.4m depth		SS	4	91	7		
End of Borehole (2.84)							

Practical refusal to augering at 2.84m depth

(Subsurface meltwater from borehole sidewalls observed at ground surface - April 2, 2020)

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Invenstigation
Proposed 3-Storey Building - 3636 Innes Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5292

REMARKS

HOLE NO. BH 3

BORINGS BY CME-55 Low Clearance Drill **DATE** March 26, 2020

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	90.08
25mm Asphaltic concrete over brown silty sand with crushed stone **FILL** (to 0.63)	SS	1	42	35		
FILL: Compact, brown silty clay, some sand and gravel (to 1.12)	SS	2	46	8	1	89.08
Very stiff to stiff, brown **SILTY CLAY**	SS	3	100	8	2	88.08
- with sand and gravel and grey by 2.6m depth (to 2.80)	SS	4	100	3		

End of Borehole

Practical refusal to augering at 2.80m depth

GWL @ 0.44m - April 2, 2020)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Invenstigation
Proposed 3-Storey Building - 3636 Innes Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5292**

REMARKS

HOLE NO. **BH 4**

BORINGS BY CME-55 Low Clearance Drill **DATE** March 26, 2020

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.94
Asphaltic concrete	0.08						
FILL: Brown silty sand with crushed stone	0.60	SS	1	36	50+		
Very stiff, brown **SILTY CLAY,** some sand	1.37	SS	2	79	9	1	88.94
GLACIAL TILL: Compact to dense, brown silty clay with sand, some gravel, cobbles and boulders - grey by 2.4m depth	2.54	SS	3	100	50+	2	87.94
		SS	4	70	50+		

End of Borehole

Practical refusal to augering at 2.54m depth

(GWL @ 0.50m - April 2, 2020)

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 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.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

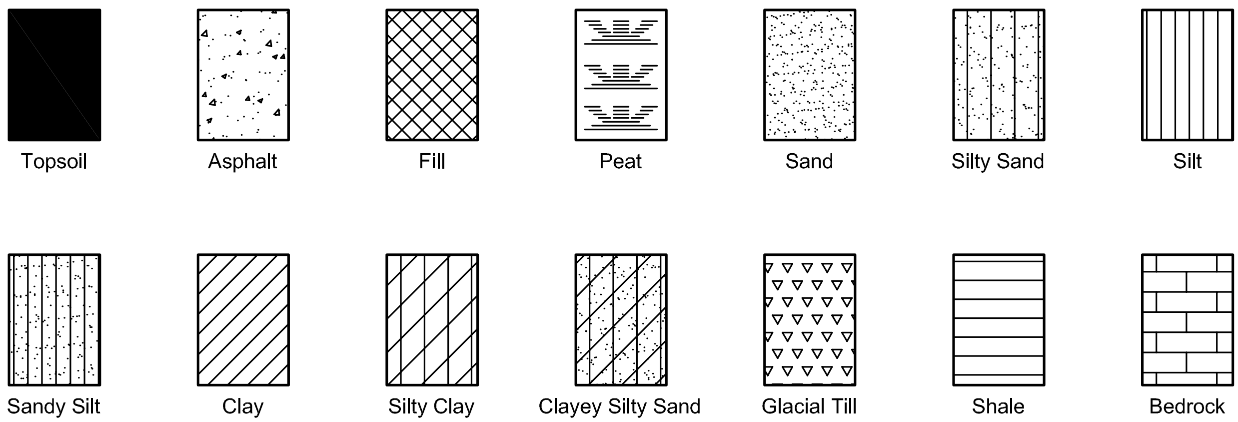
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

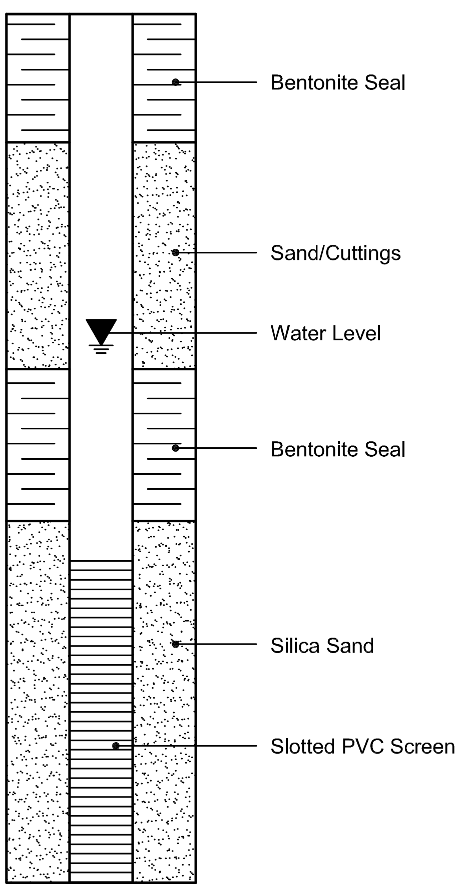
SYMBOLS AND TERMS (continued)

STRATA PLOT

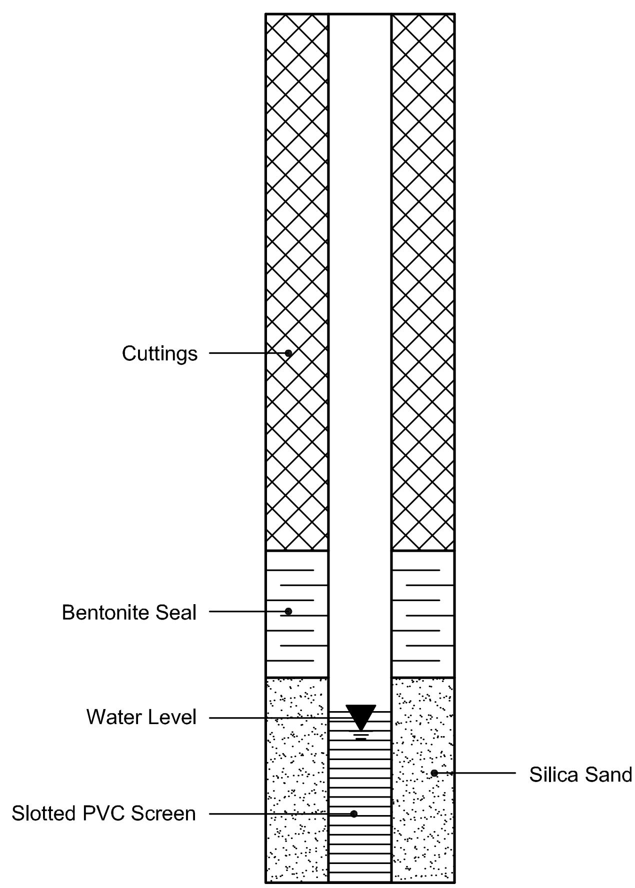


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



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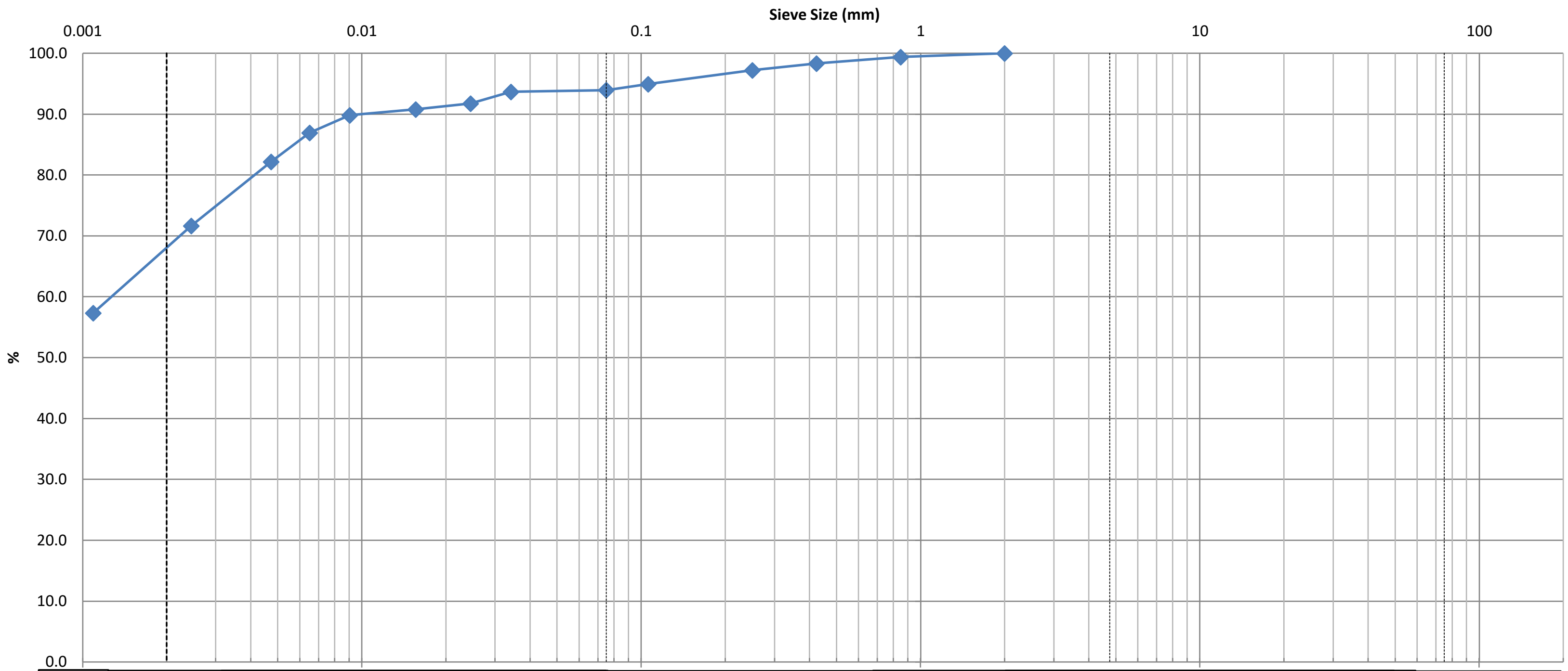
SIEVE ANALYSIS	ASTM C136 - ASTM 422

CLIENT:	Amerco Real Estate Company	DEPTH:	-	FILE NO:	PG5292
CONTRACT NO.:		BH OR TP No.:	BH2	LAB NO:	15096
PROJECT:	3 Story Building			DATE RECEIVED:	2-Apr-20
				DATE TESTED:	3-Apr-20
DATE SAMPLED:	-			DATE REPORTED:	7-Apr-20
SAMPLED BY:	A. Cooney			TESTED BY:	D.Kerr



Clay	Silt	Sand			Gravel		Cobble
		Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification	MC(%)	LL	PL	PI	Cc	Cu
		34.8					

	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
					0.0	6.1	25.4	68.5

Comments:

REVIEWED BY:	Curtis Beadow	Joe Fosyth, P. Eng.

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HYDROMETER
LS-702 ASTM-422

CLIENT:	Amerco Real Estate Company	DEPTH:	-	FILE NO.:	PG5292
PROJECT:	3 Story Building	BH OR TP No.:	BH2	DATE SAMPLED	-
LAB No. :	15096	TESTED BY:	D.Kerr	DATE RECEIVE	2-Apr-20
SAMPLED BY:	A. Cooney	DATE REPT'D:	7-Apr-20	DATE TESTED:	3-Apr-20

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
99.5	2.700

INITIAL WEIGHT	50.00	**HYGROSCOPIC MOISTURE**		
WEIGHT CORRECTED	48.40	TARE WEIGHT	50.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	3.02	AIR DRY	150.00	100.00
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	146.80	96.80
		CORRECTED	0.968	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
13.2			
9.5			
4.75			
2.0	**0.0**	0.0	**100.0**
Pan	**99.5**		
0.850	**0.29**	0.6	**99.4**
0.425	**0.81**	1.7	**98.3**
0.250	**1.35**	2.8	**97.2**
0.106	**2.46**	5.1	**94.9**
0.075	**2.93**	6.1	**93.9**
Pan	**3.02**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (°C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	8:19	55.0	6.0	23.0	0.0342	93.6	**93.6**
2	8:20	54.0	6.0	23.0	0.0245	91.7	**91.7**
5	8:23	53.5	6.0	23.0	0.0156	90.8	**90.8**
15	8:33	53.0	6.0	23.0	0.0091	89.8	**89.8**
30	8:48	51.5	6.0	23.0	0.0065	87.0	**87.0**
60	9:18	49.0	6.0	23.0	0.0047	82.2	**82.2**
250	12:28	43.5	6.0	23.0	0.0025	71.7	**71.7**
1440	8:18	36.0	6.0	23.0	0.0011	57.3	**57.3**

COMMENTS:

Moisture Content = 34.8%

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.

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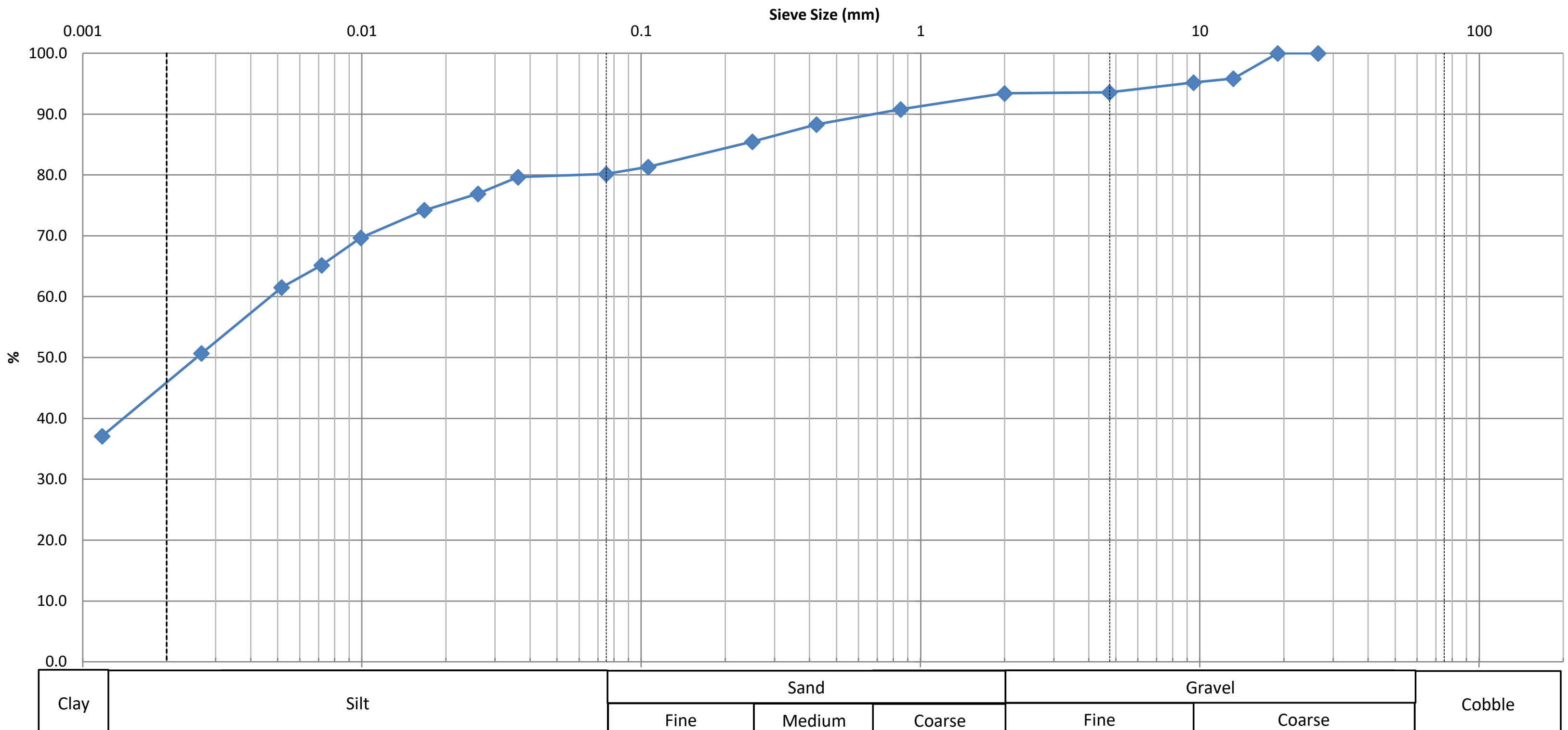
SIEVE ANALYSIS **ASTM C136**

CLIENT:	Amerco Real Estate Company	DEPTH:	-	FILE NO:	PG5292
CONTRACT NO.:		BH OR TP No.:	BH4	LAB NO:	15097
PROJECT:	3 Story Building			DATE RECEIVED:	2-Apr-20
				DATE TESTED:	3-Apr-20
DATE SAMPLED:	-			DATE REPORTED:	7-Apr-20
SAMPLED BY:	A. Cooney			TESTED BY:	D.Kerr



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					26.0					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)	Clay (%)	
					6.4	13.4		33.7	46.5	

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Fosyth, P. Eng.**

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HYDROMETER
LS-702 ASTM-422

CLIENT:	Amerco Real Estate Company	DEPTH:	-	FILE NO.:	PG5292
PROJECT:	3 Story Building	BH OR TP No.:	BH4	DATE SAMPLED	-
LAB No. :	15097	TESTED BY:	D.Kerr	DATE RECEIVE	2-Apr-20
SAMPLED BY:	A. Cooney	DATE REPT'D:	7-Apr-20	DATE TESTED:	3-Apr-20

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
122.8	2.700

INITIAL WEIGHT	50.00	**HYGROSCOPIC MOISTURE**		
WEIGHT CORRECTED	46.70	TARE WEIGHT	50.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	7.13	AIR DRY	150.00	100.00
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	143.40	93.40
		CORRECTED	0.934	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5		0.0	**100.0**
19	**0**	0.0	**100.0**
13.2	**5.1**	4.2	**95.8**
9.5	**5.9**	4.8	**95.2**
4.75	**7.9**	6.4	**93.6**
2.0	**8.1**	6.6	**93.4**
Pan	**114.7**		
0.850	**1.39**	9.2	**90.8**
0.425	**2.73**	11.7	**88.3**
0.250	**4.25**	14.5	**85.5**
0.106	**6.47**	18.7	**81.3**
0.075	**7.09**	19.8	**80.2**
Pan	**7.13**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	9:00	50.0	6.0	23.0	0.0363	85.3	**79.6**
2	9:01	48.5	6.0	23.0	0.0261	82.4	**76.9**
5	9:04	47.0	6.0	23.0	0.0167	79.5	**74.2**
15	9:16	44.5	6.0	23.0	0.0099	74.6	**69.7**
30	9:29	42.0	6.0	23.0	0.0072	69.8	**65.2**
60	9:59	40.0	6.0	23.0	0.0052	65.9	**61.5**
250	13:09	34.0	6.0	23.0	0.0027	54.3	**50.7**
1440	8:59	26.5	6.0	23.0	0.0012	39.7	**37.1**

COMMENTS:

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29578

Report Date: 02-Apr-2020

Order Date: 27-Mar-2020

Project Description: PG5292

	Client ID:	BH4-SS4	-	-	-
	Sample Date:	26-Mar-20 10:00	-	-	-
	Sample ID:	2013372-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	67.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.40	-	-	-
Resistivity	0.10 Ohm.m	20.3	-	-	-
Anions					
Chloride	5 ug/g dry	68	-	-	-
Sulphate	5 ug/g dry	351	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SHEAR WAVE VELOCITY TESTING PROFILES

DRAWING PG5292-1 - TEST HOLE LOCATION PLAN

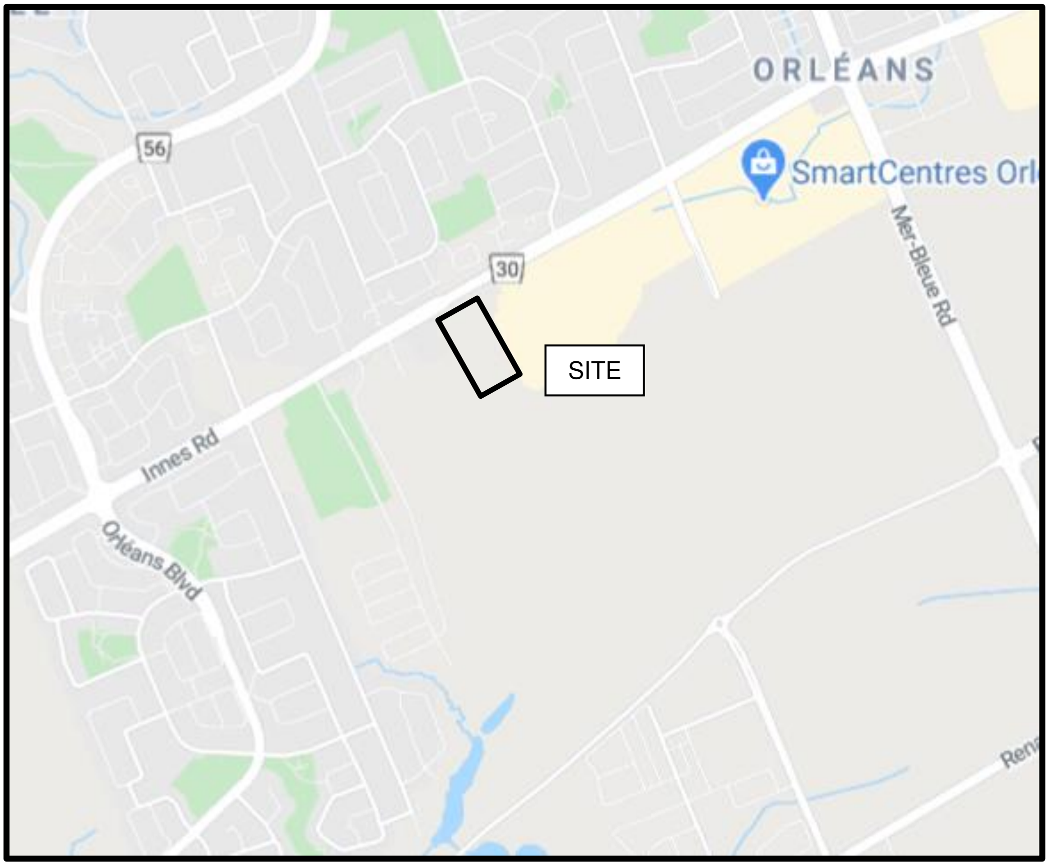


FIGURE 1

KEY PLAN

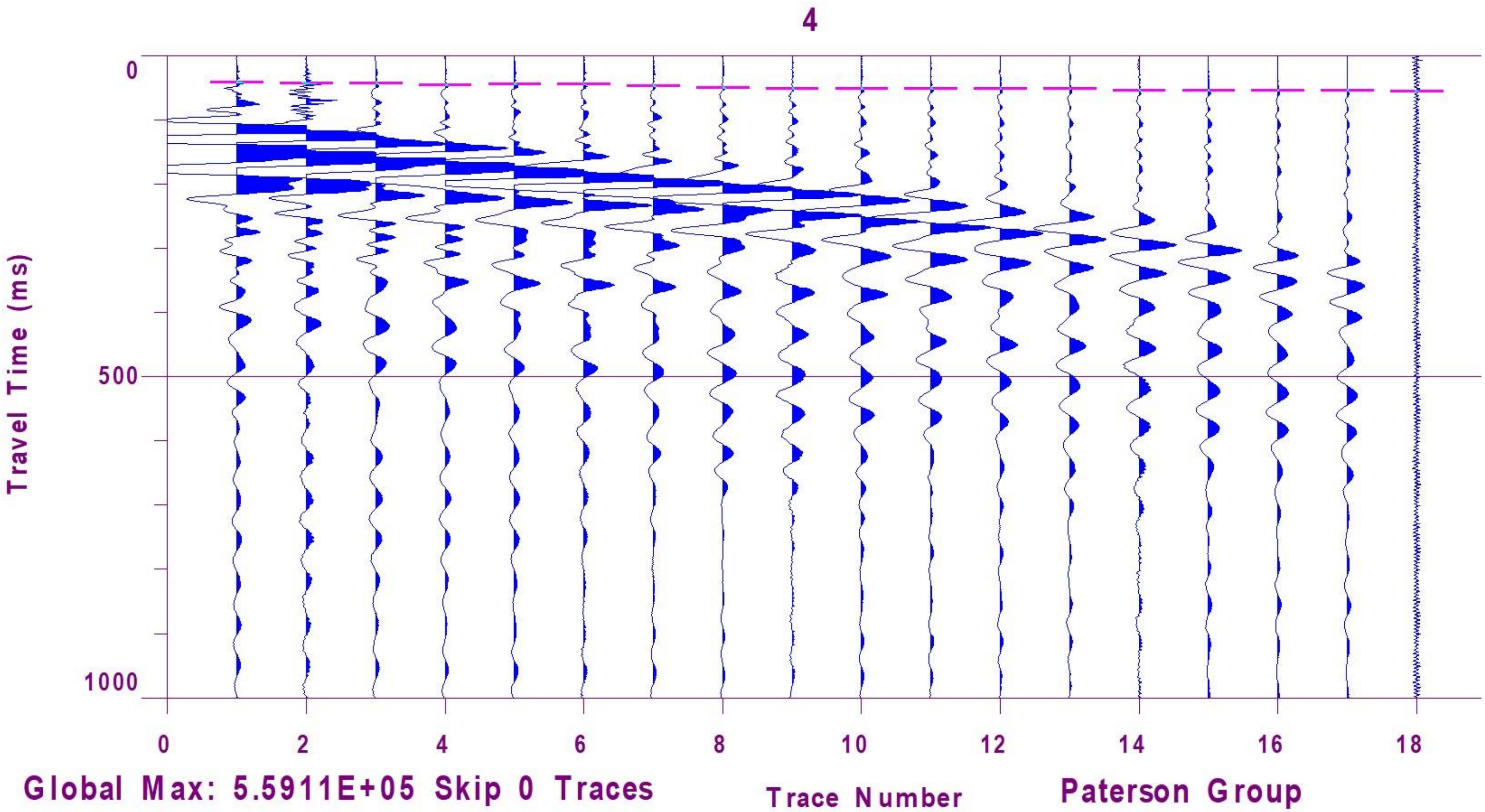


Figure 2 – Shear Wave Velocity Profile at Shot Location -25 m

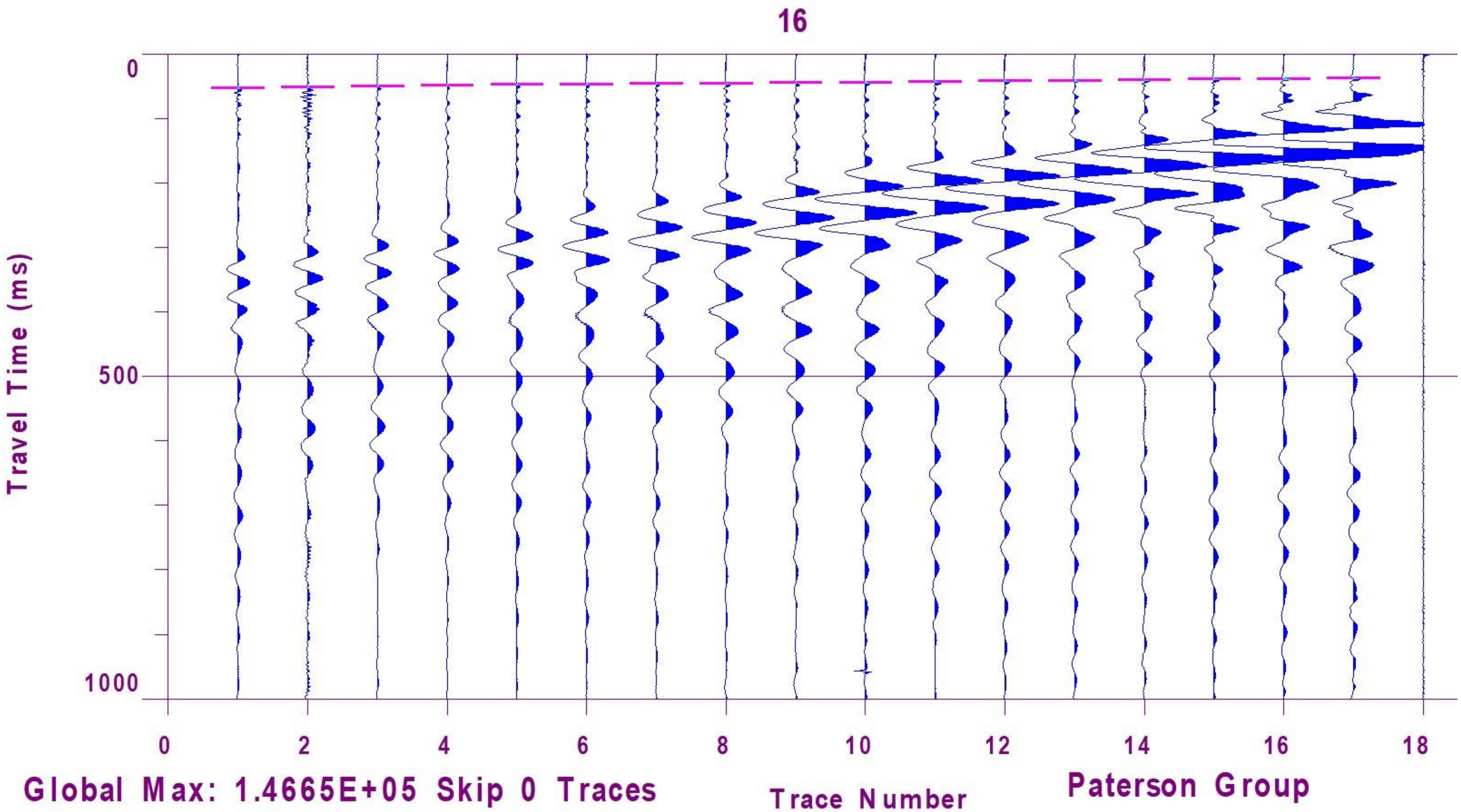
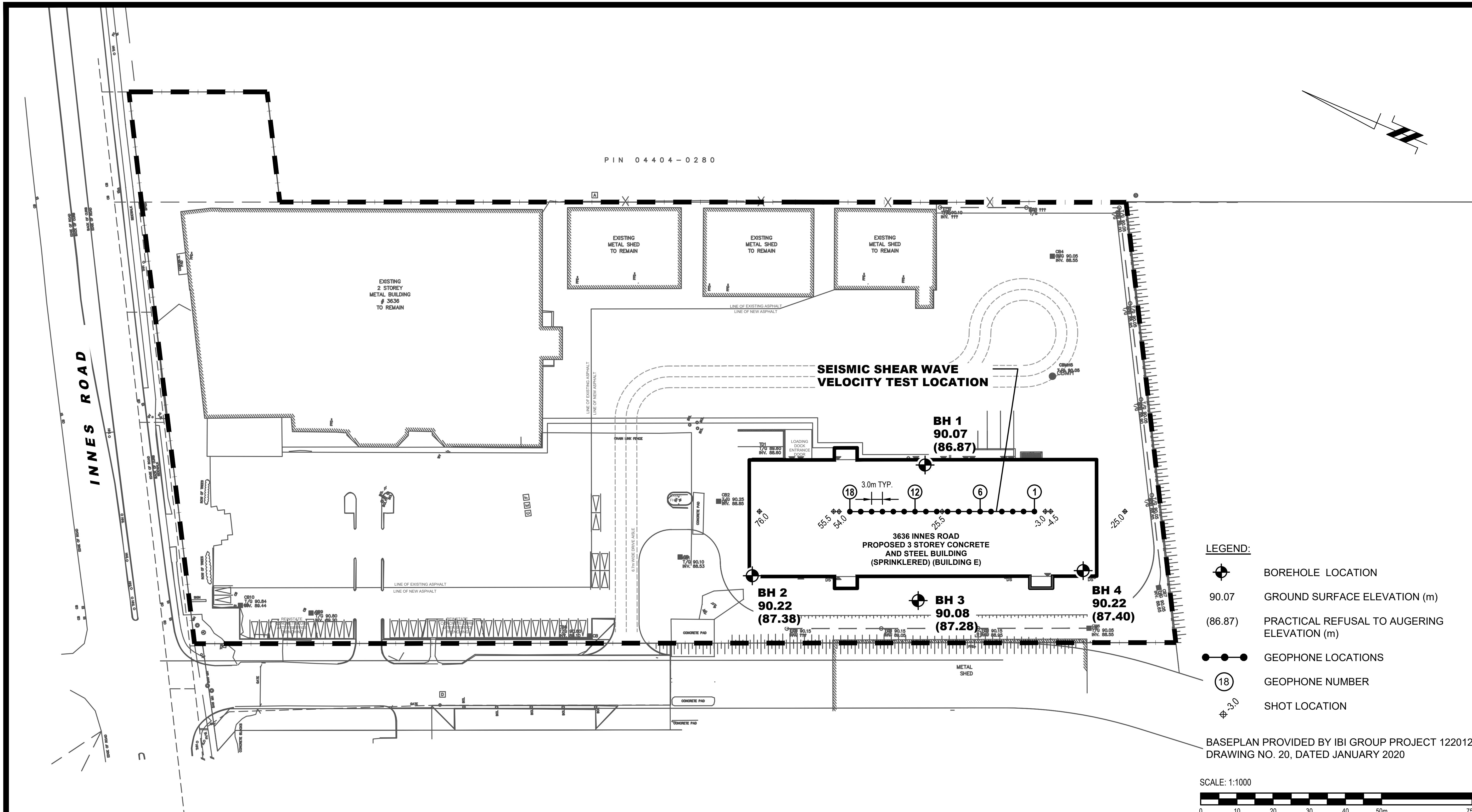


Figure 3 – Shear Wave Velocity Profile at Shot Location +76 m



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NO.	REVISIONS	DATE	INITIAL
1	SEISMIC SURVEY INFORMATION ADDED	06/22/2020	DJG

AMERCO REAL ESTATE COMPANY

GEOTECHNICAL INVESTIGATION
PROPOSED 3 STOREY U-HAUL BUILDING - 3636 INNES ROAD

OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:1000	Date: 04/2020
Drawn by: RCG	Report No.: PG5292-1
Checked by: HF	Dwg. No.: PG5292-1
Approved by: DJG	Revision No.: 1

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