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Geotechnical Investigation

Proposed Mixed-Use Building
129 Main Street
Ottawa, Ontario

Prepared For

Roderick Lahey Architect

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Report: PG2036-1 Revision 1

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road S
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Roderick Lahey Architect to conduct a geotechnical investigation for a proposed mixed-use building, which is to be located at 129 Main Street, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

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

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

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

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

2.0 Proposed Project

It is understood that the proposed project will consist of a five (5) storey residential building with one underground parking level. It is further understood that the proposed building footprint will take up the majority of the subject site.

Currently, the property is vacant and gravel covered. The subject site is bordered by a two storey building to the north, residential dwellings to the east, Springhurst Avenue to the south and Main Street to the west. It should be noted that the east portion of the north neighbouring building is founded along the subject site's north property boundary.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on March 22, 2010. At that time, three (3) boreholes were advanced to a maximum depth of 9.8 m. A supplementary investigation was completed on June 13, 2017 and consisted of excavating four (4) test pits to a maximum depth of 3.9 m below existing grade. The test hole locations were distributed in a manner to provide general coverage of the subject site. The locations of the test holes are shown on Drawing PG2036-1 - 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 whereas the test pits were excavated using a rubber tired backhoe. 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, auger samples and grab samples were recovered from the boreholes are shown as SS, AU and G, 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.

The overburden thickness was also evaluated during the course of the investigation by dynamic cone penetration testing (DCPT) at BH 2. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

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 monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The borehole locations were selected, determined in the field and surveyed by Paterson. The ground surface elevation at each borehole location was referenced to a temporary benchmark (TBM), consisting of the top of spindle of a fire hydrant, located on the west side of Main Street at Springhurst Avenue. The geodetic elevation of the TBM was surveyed to be 65.66 m, as provided by Roderick Lahey Architect. The location of the TBM and boreholes, as well as, the ground surface elevations at borehole locations are presented on Drawing PG2036-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

Currently, the subject site is gravel covered and approximately at grade with neighbouring streets. The ground surface slopes gradually down towards the south across the site.

An environmental remediation program was completed for the subject site by another environmental firm. At that time, Paterson was monitoring site activities on behalf of the property purchaser. It should be noted that due to the proximity of the east portion of the north neighbouring building to the north property line, lean concrete was poured against the existing neighbouring building's footing to provide lateral support, where suspected impacted soils were removed. The lean concrete extended to an approximately 2 to 3 m depth.

4.2 Subsurface Profile

Generally, the subsurface profile at the borehole locations consisted of imported sand and gravel fill material underlain by a native, stiff to very stiff silty clay deposit. Practical refusal to DCPT was encountered at a 26.5 m depth at BH 2.

The subsurface profile at the test pit locations consisted of a thin layer of crushed stone overlying a fill layer consisting of silty sand mixed with gravel and trace construction debris. The native silty clay deposit was encountered at TP 3 and TP 4 at depths of 3.8 and 1.9 m respectively. Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

It should be noted that the excavation sidewalls within the silty sand fill layer began collapsing once the test pits reached a depth of approximately 2 m. Based on this observation, the excavation sidewalls will not remain stable at the time of excavation down to the underside of footing elevation.

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of shale of the Billings Formation.

As previously noted, the existing north neighbouring building is partially supported with lean concrete. The concrete remains on site beneath the fill at this location. The concrete extends vertically from the top of the building footings (~2 m below surface) to a 3 to 4 m depth and horizontally from the property line to about 2.5 m away from the north property line.

4.3 Groundwater

Groundwater levels (GWLs) were measured in the standpipes installed in the boreholes and the results are summarized in Table 1. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels		Recording Date
		Depth (m)	Elevation (m)	
BH 1	64.42	3.89	60.53	March 25, 2010
BH 2	64.11	3.38	60.73	March 25, 2010
BH 3	64.86	3.96	60.90	March 25, 2010

Note: Ground surface elevations at borehole locations were referenced to a TBM, consisting of the top spindle of a fire hydrant located on the west side of Main street at Springhurst Avenue.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed building. With one (1) level of underground parking, the founding elevation is estimated to be 4 m below ground surface. It is anticipated that conventional footing foundations could be utilized. However, if design building loads are too high, consideration should be given to founding the proposed building on a raft foundation.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, containing deleterious or organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

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 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 used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

5.3 Foundation Design

Conventional Footings

Footings, up to 6 m wide, founded on undisturbed, stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS.

To allow the underside of footing to remain at the current design elevations, the footings which are presently proposed within the silty sand fill layer can be extended down to an adequate stiff silty clay bearing surface. This can be completed by excavating a zero-entry, near vertical trench extending to an undisturbed, stiff silty clay bearing surface and backfilled with a minimum 15 MPa lean concrete up to the design underside of footing elevation.

The sub-excavation required to extend the footings down to the undisturbed, silty clay bearing surface will likely require large quantities of material to be removed due to collapsing of the silty sand excavation sidewalls. Depending on the differences in elevation of the proposed footings, the proximity of adjacent footings and depth of sub-excavation required, undermining may occur. Refer to the following section for recommendations on lateral support. The bearing surfaces for all footings should be reviewed in the field by Paterson to determine the extent of sub-excavation required at the time of construction.

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 placed on an undisturbed, stiff to very stiff silty clay bearing surface and designed using the bearing resistance value at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 15 mm, respectively.

A permissible grade raise restriction of 1.5 m is recommended for the subject site.

Lateral Support

The bearing medium under footing-supported structured is required to provide adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the soil subgrade medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V, passes only through in-situ soil or engineered fill of the same or higher capacity as the soil.

Depending on the required depth of sub-excavation for adjacent footings in close proximity, the sub-excavation for one footing may undermine the lateral support of the adjacent footing. If sub-excavation causes undermining, the undermined footing should be sub-excavated to an elevation with adequate lateral support, and the sub-excavated portion below the underside of footing should be backfilled with lean-mix concrete as described above. This should be reviewed by Paterson in the field at the time of excavation to ensure undermining does not occur.

Raft Foundation

Consideration can be given to a raft foundation if the building loads exceed the bearing resistance values given above. The following parameters may be used for raft design.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. A bearing resistance value at SLS (contact pressure) of **150 kPa** can be used. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The factored bearing resistance (contact pressure) at ULS can be taken as **275 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **6 MPa/m** for a contact pressure of **150 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Based on the following assumptions for the raft foundation, the proposed building can be designed using the above parameters and a total and differential settlement of 25 and 15 mm, respectively. It is expected that the base of the slab is located at or below 4 m depth, the long term groundwater level will be at or below 4 m depth, the raft slab is impervious and the basement walls will be provided with a perimeter foundation drainage system.

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 existing building from Table 4.1.8.4.A of the Ontario Building Code 2012. Our shear wave velocity calculations take into consideration an assumed underside of footing depth of 4 m. The shear wave velocity testing was completed by Paterson personnel. Two (2) shear wave velocity profiles from our seismic testing are attached to Appendix 2.

Field Program

The shear wave testing was located within the proposed building footprint, as presented in Drawing PG2036-1 - Test Hole Location Plan attached to the present letter. Paterson field personnel placed 20 horizontal geophones in a straight line in roughly a northeast-southwest 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 2 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a laptop computer 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 were located at the centre of the geophone array and 2, 3 and 22 m away from the first and last geophone.

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

Data Processing and Interpretation

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

Based on our analysis of the shear wave velocity profiles, the average shear wave velocity through the overburden soil was interpreted to be **197 m/s**.

Based on our testing results, a bedrock velocity could not be accurately defined. In lieu of a well defined, on-site bedrock velocity, we used bedrock shear wave velocities observed at shear wave testing locations over bedrock of the same formation with an equal or greater degree of weathering and fracturing.

Based on bedrock mapping, shale bedrock of the Billings formation is present below the subject site. Shale bedrock of the Billings formation tested by Paterson at other sites had shear wave velocities of 2,100, 1,958, 1,895 and 1,591 m/s. Bedrock was located near surface at several of these sites, where a greater degree of weathering would occur due to exposure to weathering effects, such as freeze/thaw cycles. A conservative shear wave velocity estimate of **1,500 m/s** for the bedrock will be used in our calculations.

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

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

$$V_{s30} = \frac{30m}{\left(\frac{(26.5-5.5)m}{197m/s} + \frac{9m}{1{,}500}\right)}$$

$$V_{s30} = 266m/s$$

Based on the results of the seismic testing , the average shear wave velocity of the upper 30 m profile below the proposed underside of foundation, Vs_{30}, is **266 m/s**. Therefore, a **Site Class D** is applicable for design of the proposed building as per Table 4.1.8.4.A of the OBC 2012.

5.5 Basement Slab

It is expected that the basement area will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level are anticipated where a concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab fill consists 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 at least 95% of its SPMDD.

5.6 Basement Wall

It is expected that the basement walls of the proposed building will be poured directly against the temporary shoring system. There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, in our opinion, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

It is expected that the foundation will be provided with a perimeter drainage system; therefore, the retained soils should be considered drained. However, if undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight. 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_A) 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:

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

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

Car only parking areas and access lanes are anticipated for the proposed building, the pavement structures presented in Tables 2 and 3 would be applicable for design.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 3 - Recommended Pavement Structure - Access Lanes

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

Perimeter Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. It is expected that the basement walls of the proposed building will be poured directly against the temporary shoring system.

It is recommended that the composite drainage system such as Delta Drain 6000 or equivalent, be placed between the shoring face and concrete wall and extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

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

6.2 Protection of Footings Against Frost Action

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

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

6.3 Excavation Side Slopes and Temporary Shoring

At this site, temporary shoring will be required to complete the required excavations. However, it is recommended that where sufficient room is available, open cut excavation in combination with temporary shoring can be used.

It is understood that consideration is being given to completing the excavation to the underside of footing elevation along Main Street without using a temporary shoring system. Based on the observations during the test pit investigation, significant collapse of the excavation sidewalls was noted within the upper portion of the subsurface profile (silty sand fill layer). Based on this observation, the excavation side slopes would likely need to be cut back at a very shallow slope to maintain stability. However, based on the proximity of the excavation to the property line, it is anticipated that there will be insufficient space to allow the excavation of a shallow excavation side slope. Therefore, it is recommended to install a temporary shoring system at locations where the excavation is in close proximity to the property line.

Excavation Side Slopes

The subsoil at this site is considered to be mainly a Type 2 or Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. 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 the groundwater level.

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.

Temporary Shoring

For preliminary design purposes, the temporary system may consist of a soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below.

It is important to note that the excavation for the proposed building is expected to remove lateral support of the adjacent building footings. Therefore, a temporary shoring system, such as soldier piles and lagging, should be designed to provide the necessary lateral support for the adjacent foundations. In addition, the footings of the north neighbouring building could be supported with structural brackets designed by a qualified engineer, extended under the footings and welded to the back of the soldier piles.

These systems can be cantilevered, anchored or braced. Earth pressures acting on the shoring system may be calculated using the parameters provided in Table 4.

Table 4 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Effective (undrained) Unit Weight(γ), kN/m^3	13

Soldier Pile and Lagging System

The earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of $0.65 \cdot K \cdot \gamma \cdot H$ for strutted or anchored shoring, or a triangular earth pressure distribution with a maximum value of $K \cdot \gamma \cdot H$ for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the undrained unit weights are used for earth pressure calculations, should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used for the full height, with no hydrostatic groundwater pressure component.

A minimum factor of safety of 1.5 should be used.

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. 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 to moderate 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 and Climate Change (MOECC) 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 MOECC.

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 MOECC review of the PTTW application.

6.6 Winter Construction

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

The subsoil conditions at this site 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.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

Precaution must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.7 Corrosion Potential and Sulphate

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

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 geotechnical aspects of the excavating contractor’s shoring design, prior to construction.

❑ 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 soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification 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 Roderick Lahey Architect 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.

LICENSED PROFESSIONAL ENGINEER
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Report Distribution:

☐ Roderick Lahey Architect (3 copies)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buiding - 129 Main Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on Springhurst Avenue. Geodetic elevation = 64.54m.

REMARKS

BORINGS BY Backhoe

DATE June 13, 2017

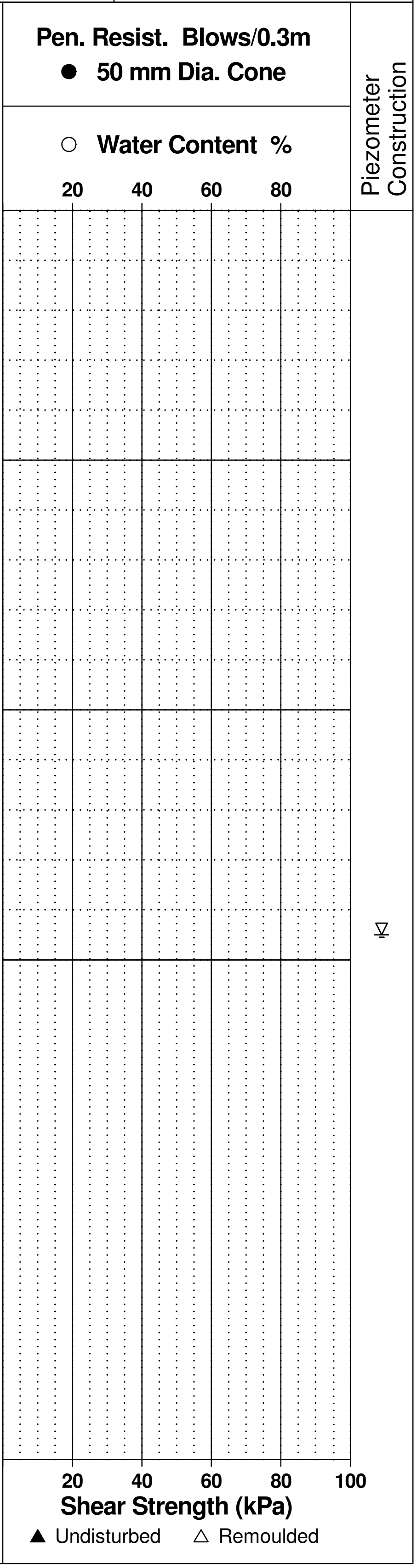
FILE NO. PG2036

HOLE NO. TP 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.89
FILL: Crushed stone with silt, trace organics 0.15	▨						
FILL: Brown silty sand, trace gravel	▨	G	1				
						1	63.89
		G	2				
						2	62.89
		G	3				
3.00		G	4				
End of Test Pit						3	61.89

Test pit sidewalls began collapsing at 2.5m depth. TP terminated at 3.00m depth due to sidwall failure.

(GWL @ 2.9m depth based on field observations)



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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buiding - 129 Main Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on Springhurst Avenue. Geodetic elevation = 64.54m.

REMARKS

BORINGS BY Backhoe

DATE June 13, 2017

FILE NO. PG2036

HOLE NO. TP 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.49
FILL: Crushed stone with silt 0.20		G	1				
FILL: Brown silty sand 0.27		G	2				
FILL: Brown silty sand, some gravel 0.41 trace construction debris		G	3				
		G	4			1	63.49
FILL: Brown silty sand, trace gravel							
		G	5			2	62.49
		G	6				
3.00						3	61.49
Stiff, grey **SILTY CLAY,** some sand		G	7				
3.30		G	8				

End of Test Pit

Test pit sidewalls began collapsing at 2.0m depth. TP terminated at 3.30m due to sidewall failure.

(TP dry upon completion)

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

○ **Water Content %**
20 40 60 80

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

Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buiding - 129 Main Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on Springhurst Avenue. Geodetic elevation = 64.54m.

REMARKS

BORINGS BY Backhoe

DATE June 13, 2017

FILE NO. PG2036

HOLE NO. TP 3

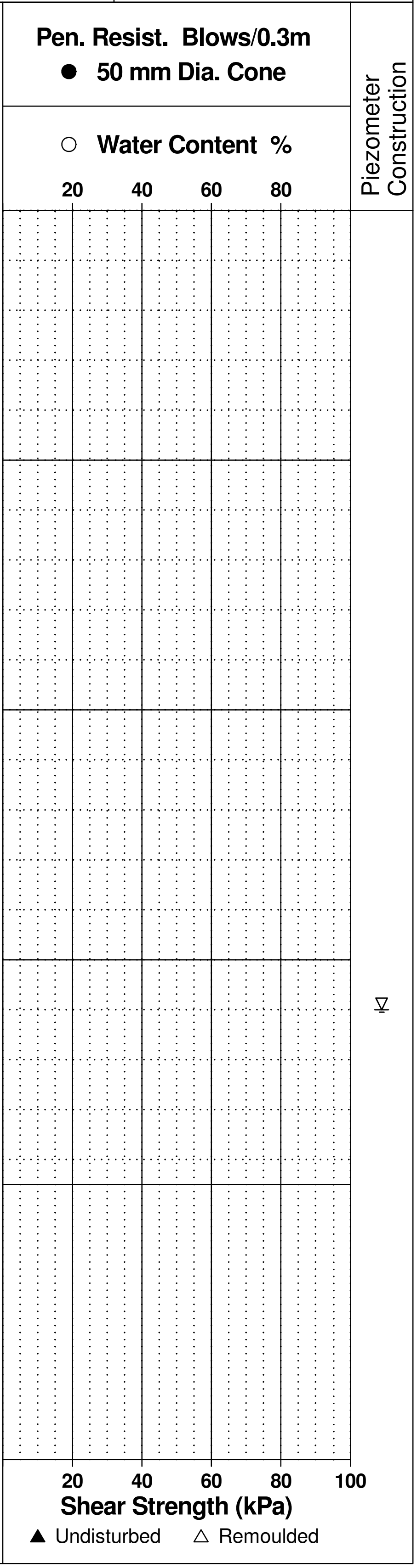
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.71
FILL: Crushed stone with silt, trace organics 0.23		G	1				
FILL: Brown silty sand, trace gravel		G	2			1	63.71
		G	3				
1.80							
FILL: Grey silty clay, some sand and gravel, trace glass, concrete, brick, metal		G	4			2	62.71
		G	5				
		G	6			3	61.71
		G	7				
3.80							
Stiff, grey **SILTY CLAY** 3.90		G	8				

End of Test Pit

Minor test pit sidwall collapse within the silty sand layer

TP terminated in silty clay 3.90m depth

(GWL @ 3.2m depth based on field observations)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Buiding - 129 Main Street
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located on Springhurst Avenue. Geodetic elevation = 64.54m.

REMARKS

BORINGS BY Backhoe

DATE June 13, 2017

FILE NO. PG2036

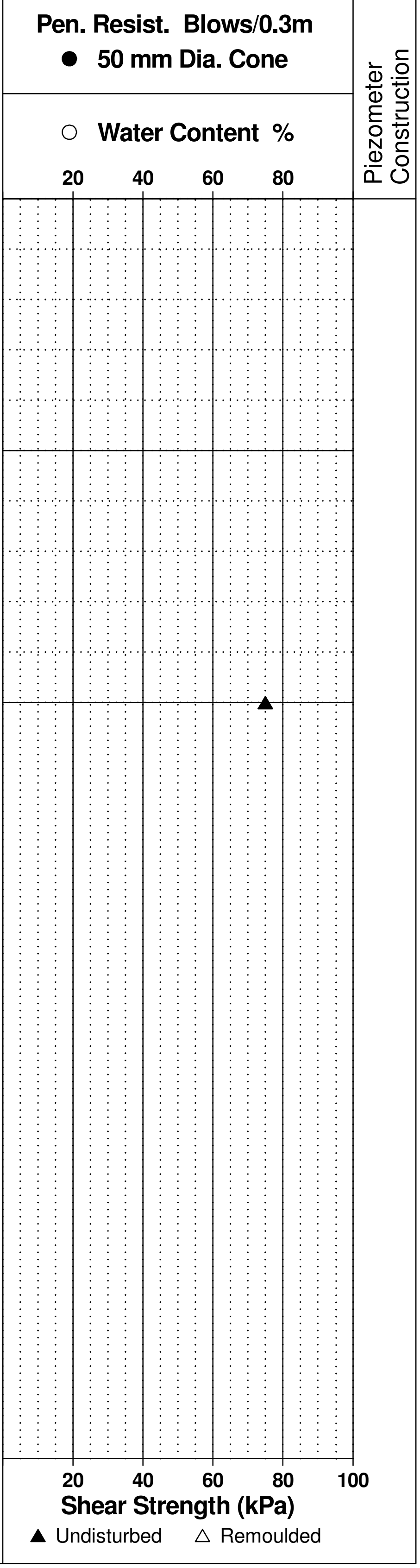
HOLE NO. TP 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.92
FILL: Crushed stone with silt, trace organics 0.21		G	1				
FILL: Brown silty sand, trace gravel 1.06		G	2			1	63.92
FILL: Grey-brown silty clay, some sand and gravel, trace construction debris 1.93		G	3				
Stiff, brown **SILTY CLAY,** some sand 2.00		G	4			2	62.92

End of Test Pit

TP terminated in silty clay at 2.00m depth

(TP dry upon completion)



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed 6-Storey Residential Building-129 Main St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, located on the west side of Main St. at Springhurst Ave. Geodetic elevation = 65.66m.

REMARKS

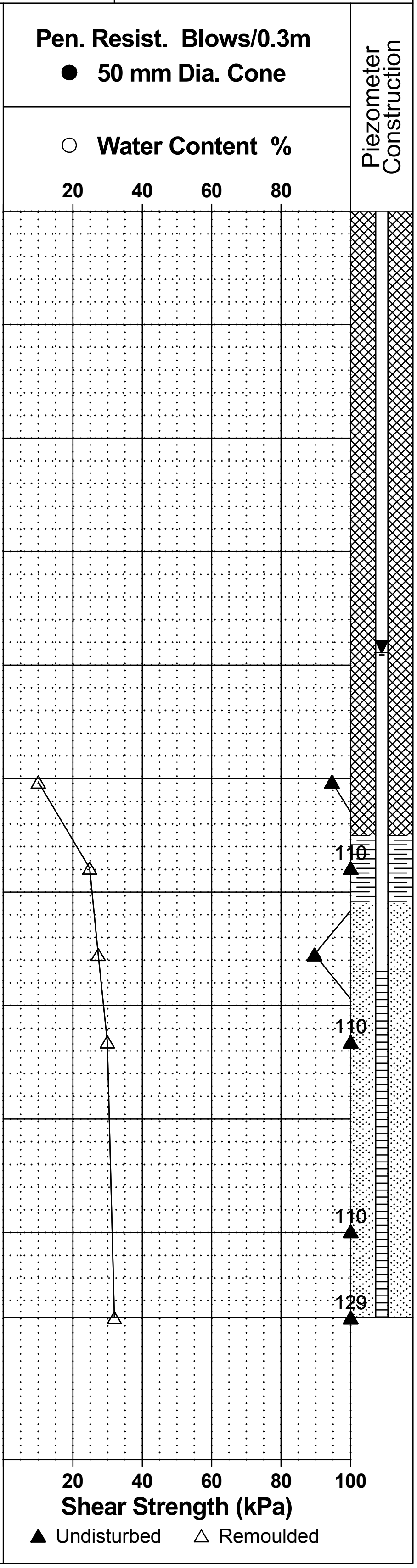
BORINGS BY CME 75 Power Auger

DATE March 22, 2010

FILE NO. **PG2036**

HOLE NO. **BH 1**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
GROUND SURFACE						0	64.42
FILL: Grey silty sand and gravel 0.15		AU	1				
		SS	2	46	22	1	63.42
FILL: Brown silty sand, trace gravel		SS	3	58	20	2	62.42
		SS	4	62	5	3	61.42
3.66		SS	5	42	4		
		SS	6	100	2	4	60.42
						5	59.42
Stiff to very stiff, dark grey **SILTY CLAY** with black organic mottling						6	58.42
						7	57.42
		SS	7		2	8	56.42
						9	55.42
9.75							
End of Borehole (GWL @ 3.89m-Mar. 25/10)							



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed 6-Storey Residential Building-129 Main St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, located on the west side of Main St. at Springhurst Ave. Geodetic elevation = 65.66m.

REMARKS

BORINGS BY CME 75 Power Auger

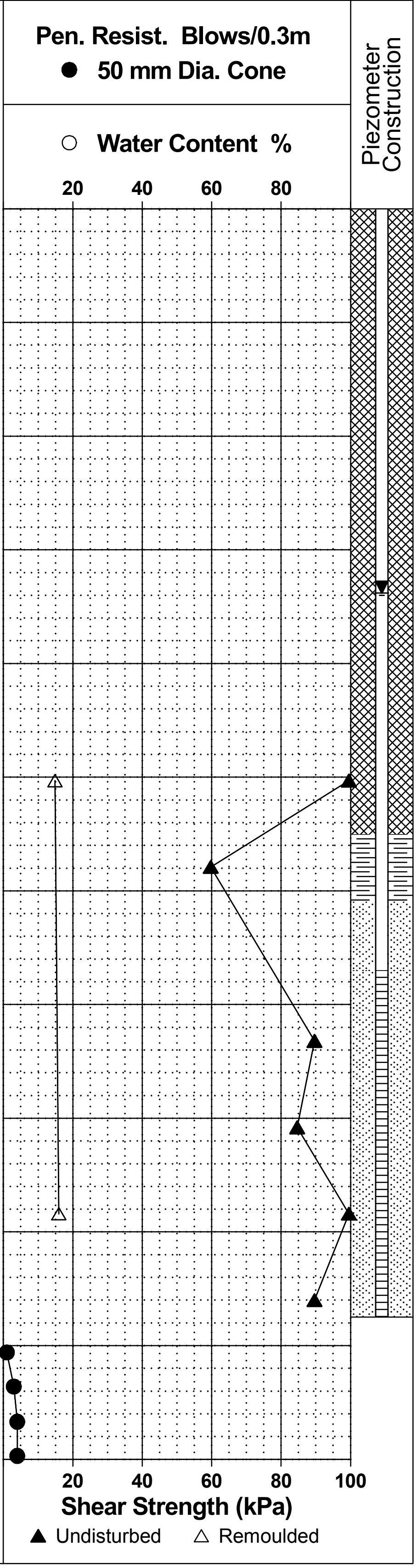
DATE March 22, 2010

FILE NO. PG2036

HOLE NO. BH 2

SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Grey silty sand and gravel (0.15)	AU	1		
FILL: Brown silty sand, trace gravel	SS	2	50	31
	SS	3	50	20
	SS	4	58	10
(3.66)	SS	5	25	7
Stiff, dark grey **SILTY CLAY**	SS	6	100	2
	SS	7	100	2
(9.75)	SS	8	100	
Dynamic Cone Penetration Test commenced @ 9.75m depth				

DEPTH (m)	ELEV. (m)
0	64.11
1	63.11
2	62.11
3	61.11
4	60.11
5	59.11
6	58.11
7	57.11
8	56.11
9	55.11
10	54.11
11	53.11



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed 6-Storey Residential Building-129 Main St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, located on the west side of Main St. at Springhurst Ave. Geodetic elevation = 65.66m.

REMARKS

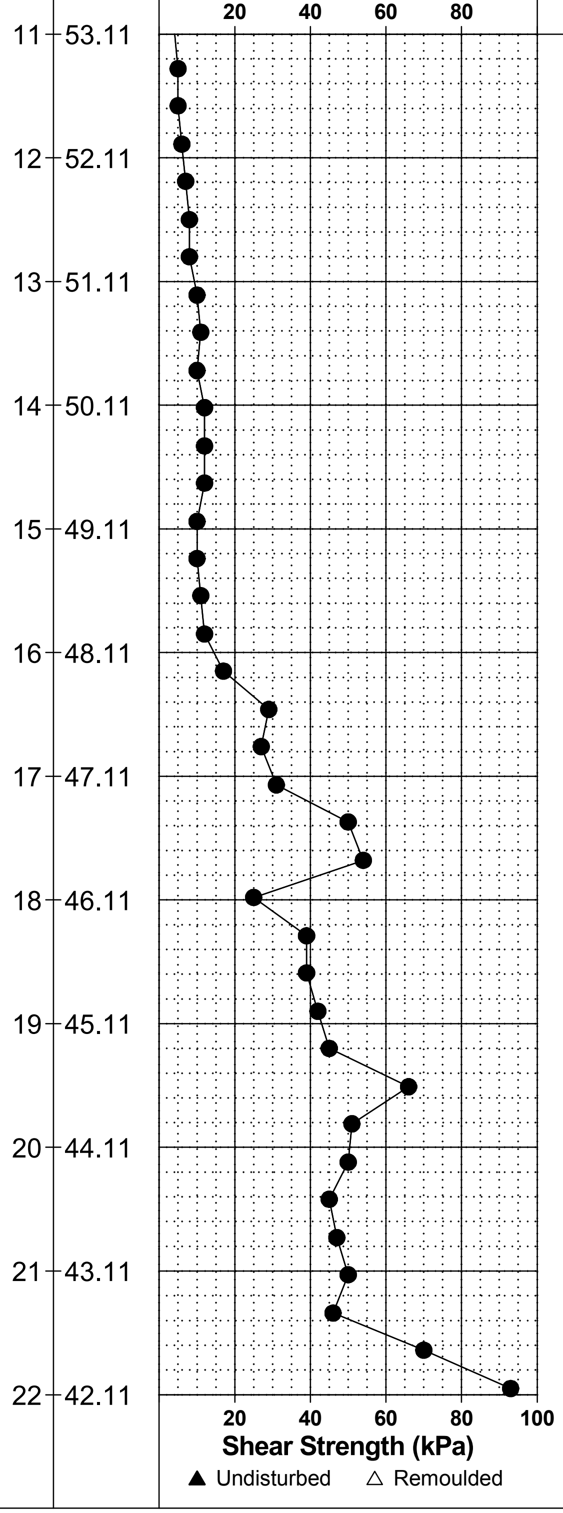
BORINGS BY CME 75 Power Auger

DATE March 22, 2010

FILE NO. PG2036

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 %	Piezometer Construction
GROUND SURFACE									



patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed 6-Storey Residential Building-129 Main St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, located on the west side of Main St. at Springhurst Ave. Geodetic elevation = 65.66m.

REMARKS

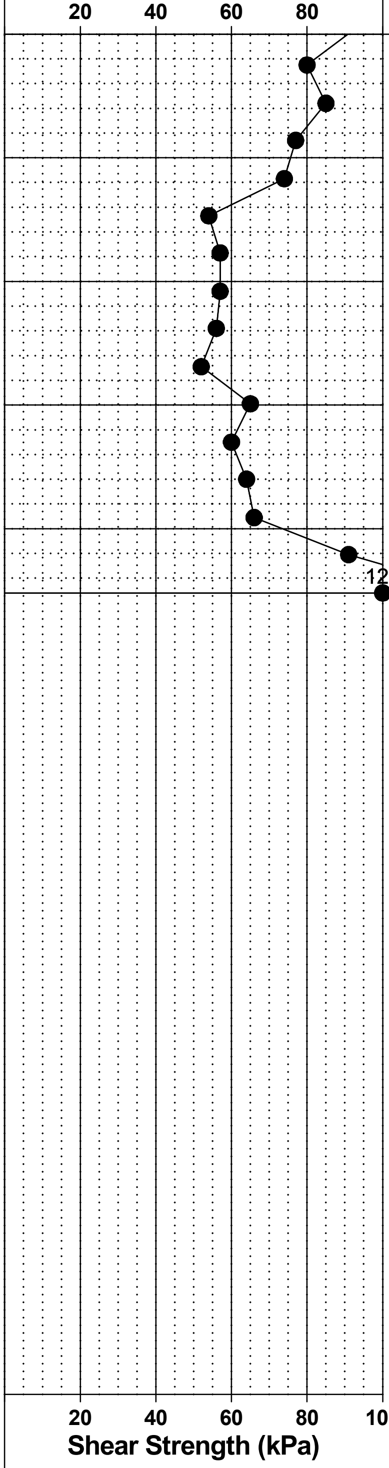
BORINGS BY CME 75 Power Auger

DATE March 22, 2010

FILE NO. PG2036

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 %	Piezometer Construction
GROUND SURFACE									
26.52						22	42.11		
						23	41.11		
						24	40.11		
						25	39.11		
						26	38.11		



End of Borehole

Practical refusal to DCPT @ 26.52m depth

(GWL @ 3.38m-Mar. 25/10)

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed 6-Storey Residential Building-129 Main St.
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, located on the west side of Main St. at Springhurst Ave. Geodetic elevation = 65.66m.

REMARKS

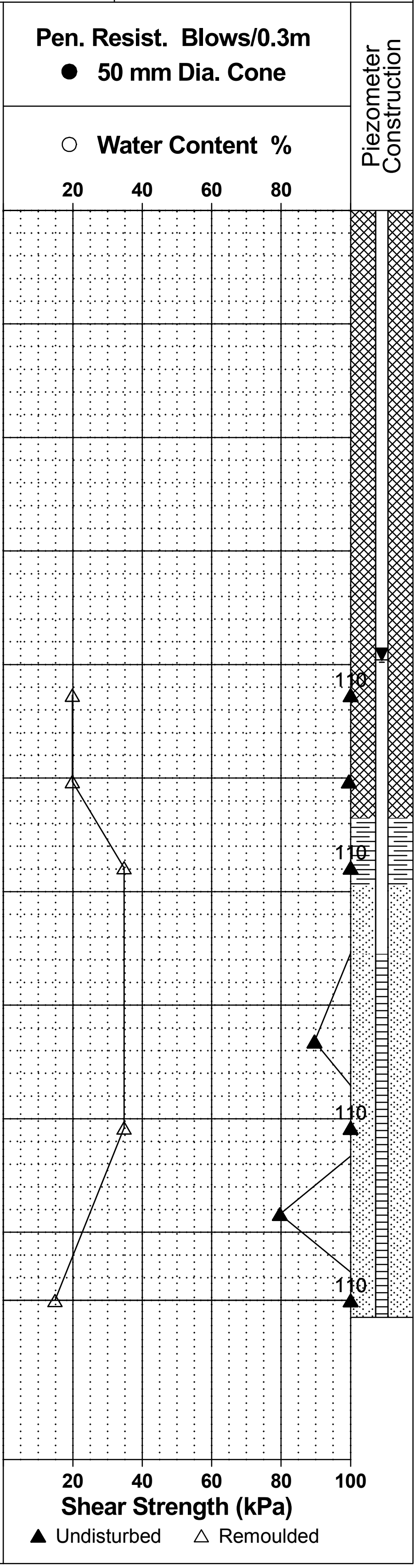
BORINGS BY CME 75 Power Auger

DATE March 22, 2010

FILE NO. **PG2036**

HOLE NO. **BH 3**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.86
FILL: Grey silty sand and gravel 0.15		AU	1				
FILL: Brown silty sand, trace gravel		SS	2	46	9	1	63.86
		SS	3	38	9	2	62.86
2.90		SS	4	21	17		
Very stiff to stiff, dark grey **SILTY CLAY**		SS	5	83	3	3	61.86
						4	60.86
						5	59.86
		SS	6	100	3	6	58.86
						7	57.86
						8	56.86
						9	55.86
9.60							
End of Borehole (GWL @ 3.96m-Mar. 25/10)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2$ / (D10 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)

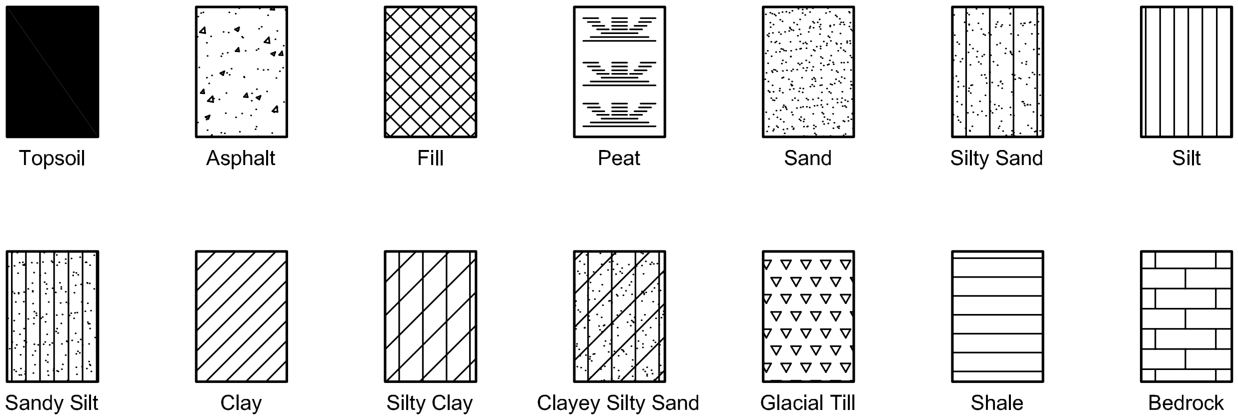
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

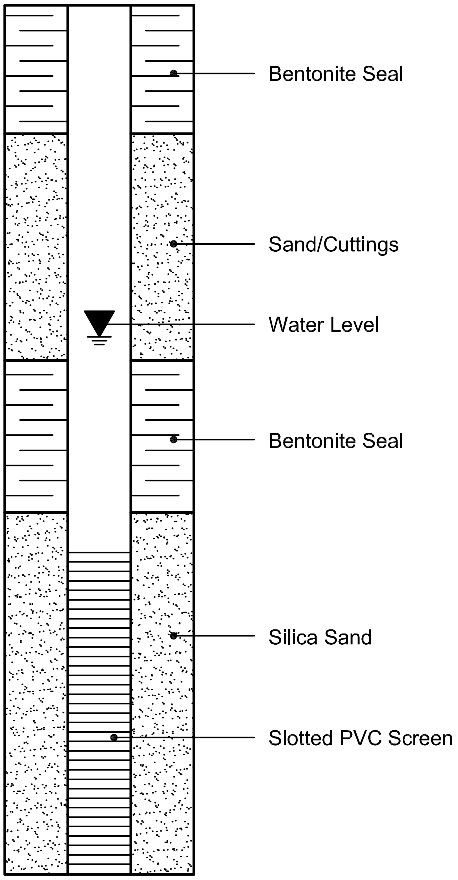
SYMBOLS AND TERMS (continued)

STRATA PLOT

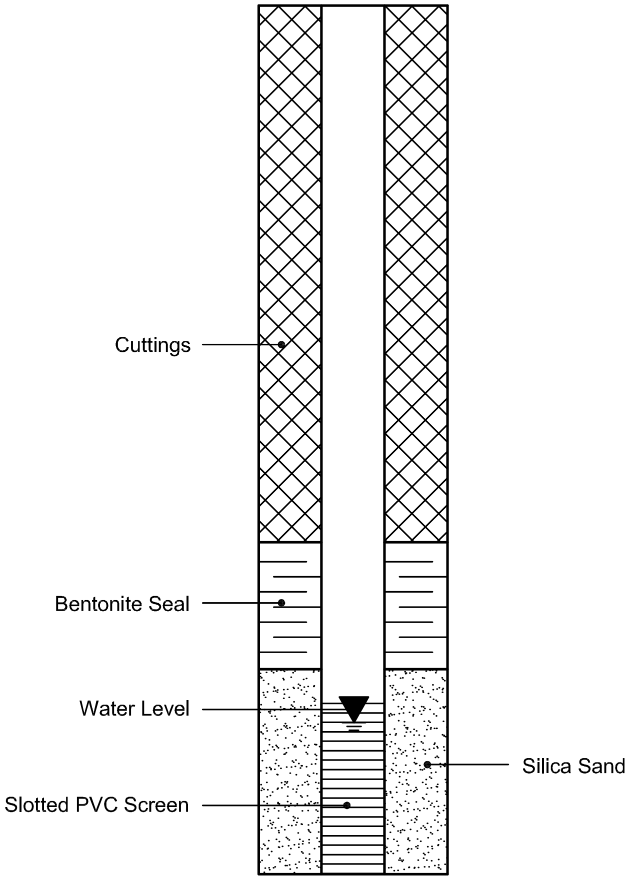


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Order #: 1013069

Certificate of Analysis

Report Date: 29-Mar-2010
Order Date:23-Mar-2010

Client: **Paterson Group Consulting Engineers**
Client PO: 8651 Project Description: PG2036

	Client ID:	BH3 SS6	-	-	-
	Sample Date:	22-Mar-10	-	-	-
	Sample ID:	1013069-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	79.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.60	-	-	-
Resistivity	0.10 Ohm.m	46.4	-	-	-
Anions					
Chloride	5 ug/g dry	21	-	-	-
Sulphate	5 ug/g dry	24	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 AND 3 - SHEAR WAVE VELOCITY PROFILES

DRAWING PG2036-1 - TEST HOLE LOCATION PLAN

Source: Bing Maps

FIGURE 1

KEY PLAN

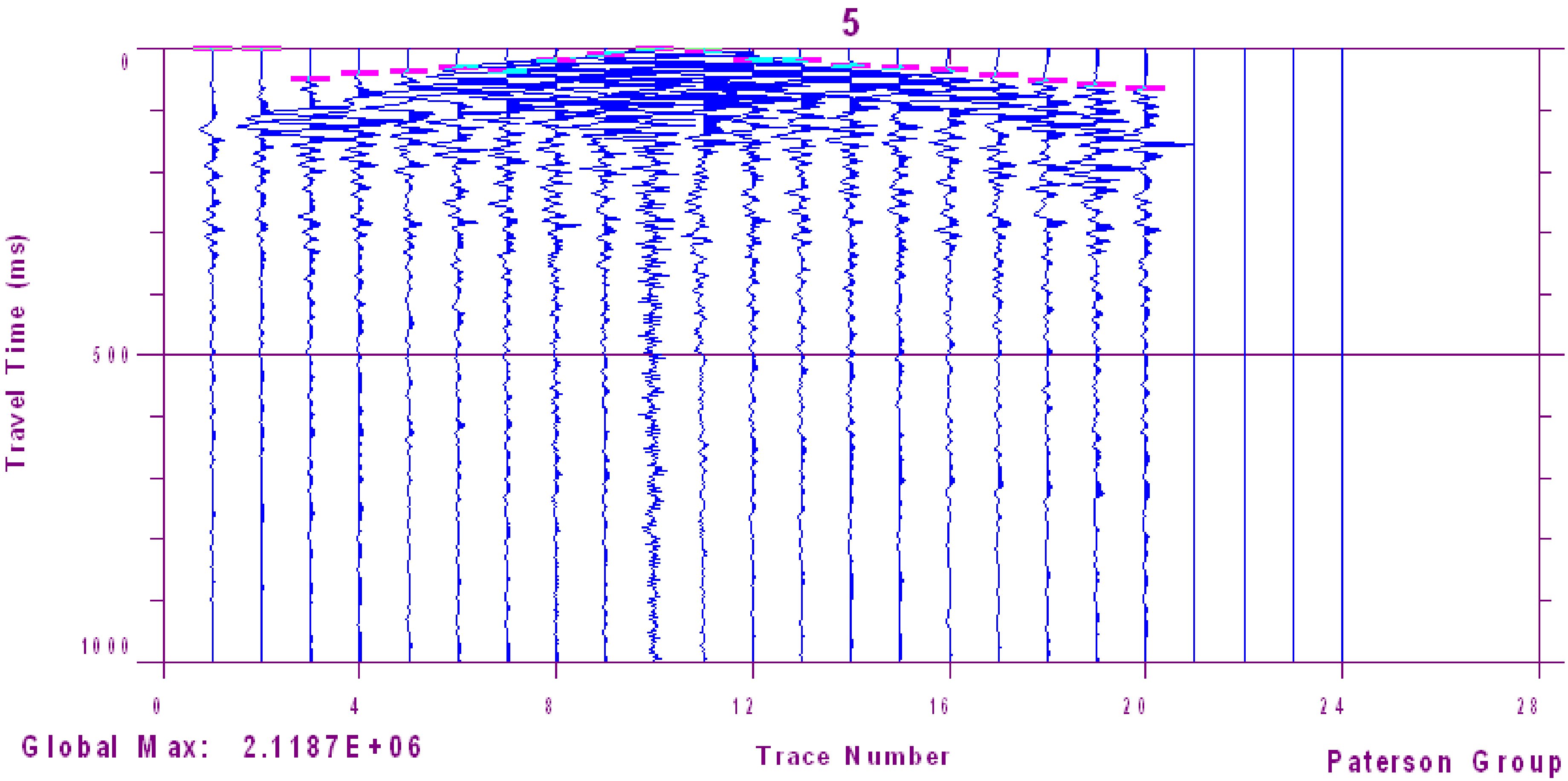


Figure 2 – Shear Wave Profile at Shot Location 19.0 m

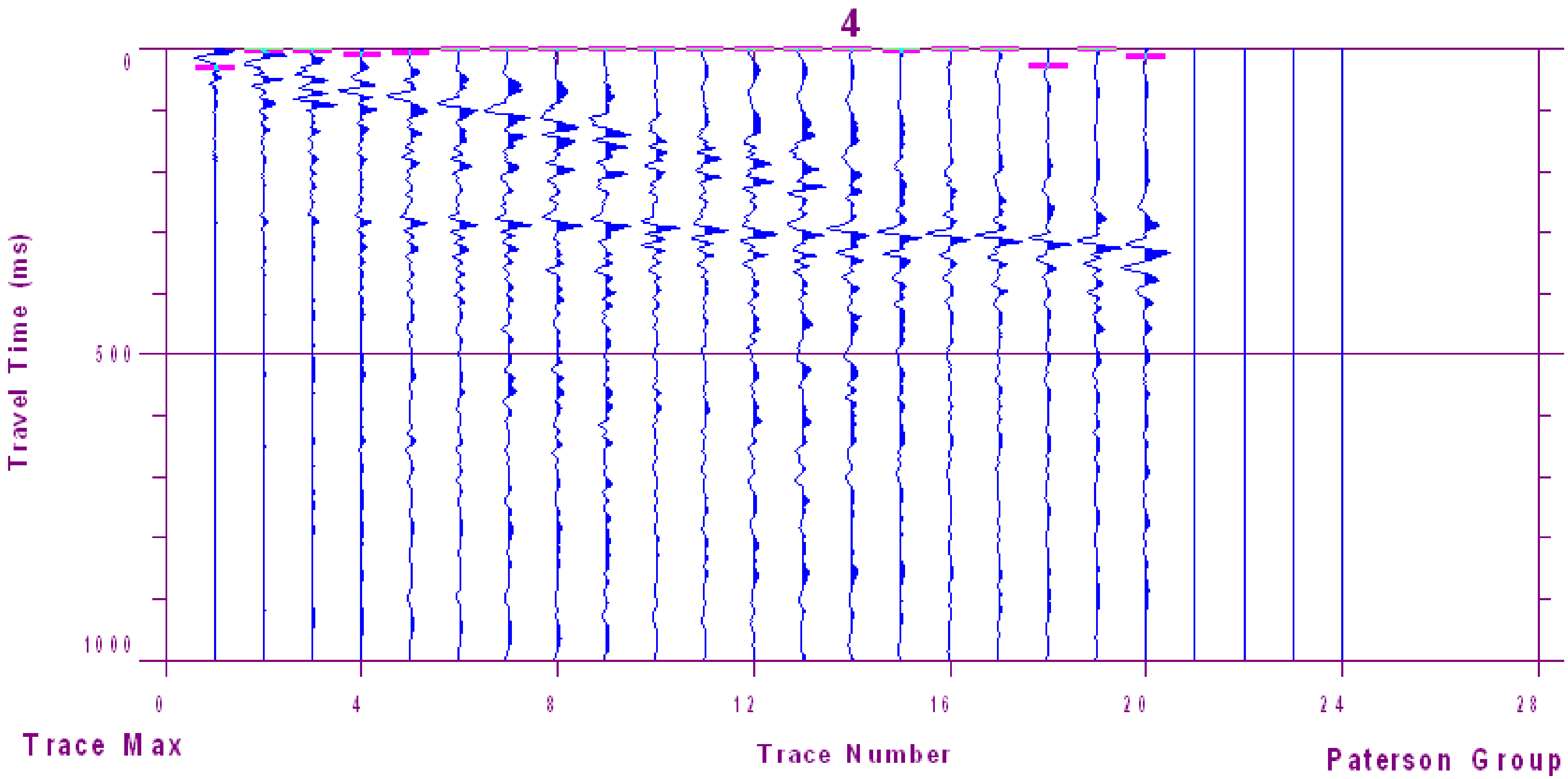
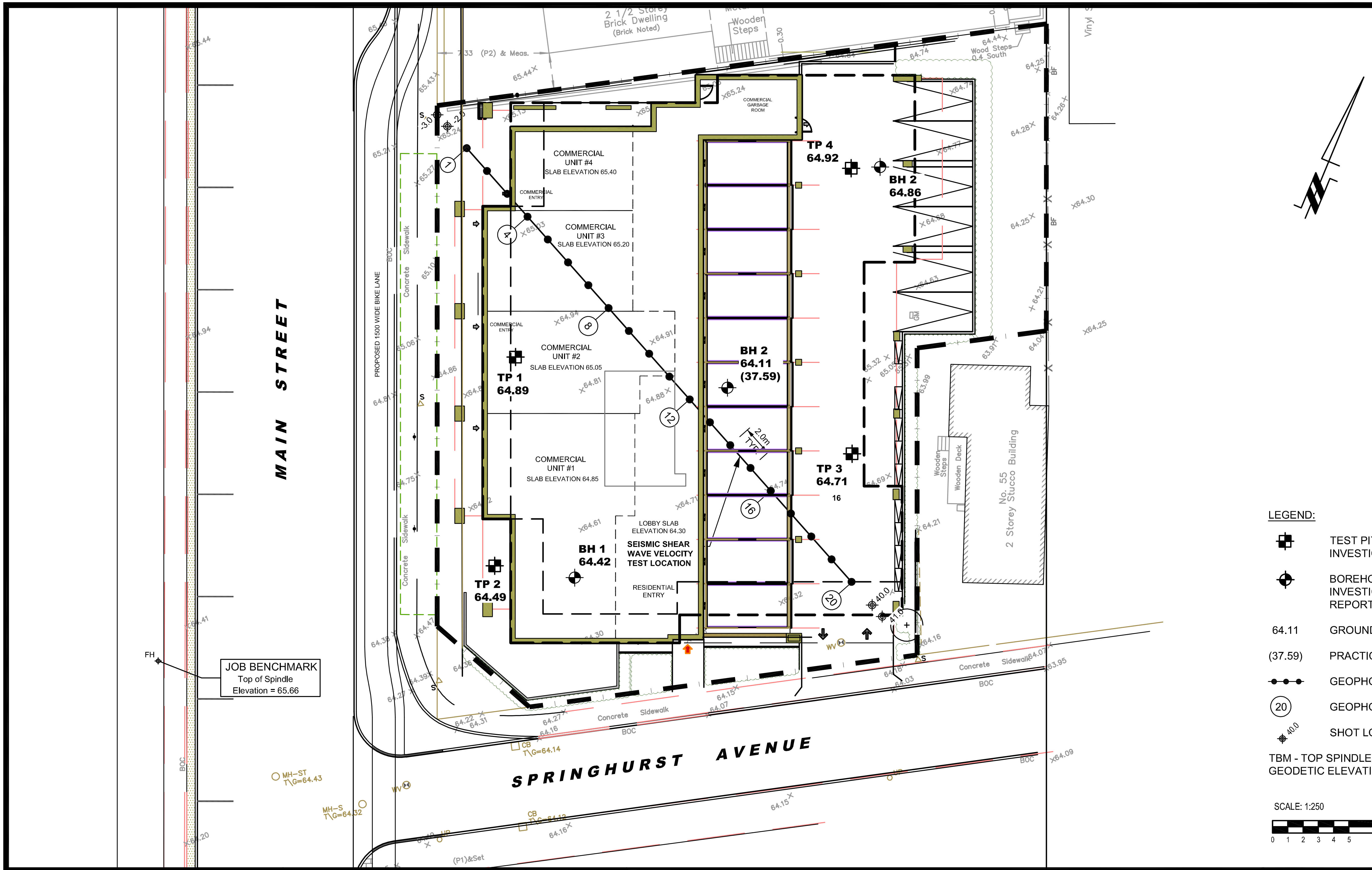


Figure 3 – Shear Wave Profile at Shot Location -2.0 m



LEGEND:

⊞	TEST PIT LOCATION, CURRENT INVESTIGATION
⊕	BOREHOLE LOCATION, PREVIOUS INVESTIGATION, PATERSON GROUP REPORT PG2036-1, 2010
64.11	GROUND SURFACE ELEVATION (m)
(37.59)	PRACTICAL DCPT REFUSAL ELEVATION (m)
●-●-●	GEOPHONE LOCATION
⑳	GEOPHONE NUMBER
◈ 40.0	SHOT LOCATION

TBM - TOP SPINDLE OF FIRE HYDRANT.
GEODETIC ELEVATION = 65.66m.

SCALE: 1:250

0 1 2 3 4 5 10 15m

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
0	BASE PLAN UPDATED	25/06/2018	DJG

RODERICK LAHEY ARCHITECT INC.
GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE BUILDING - 129 MAIN STREET
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:250
Date: 07/2017
Drawn by: MPG
Report No.: PG2036-2
Checked by: CB
Dwg. No.: PG2036-2
Approved by: DJG
Revision No.: 1

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