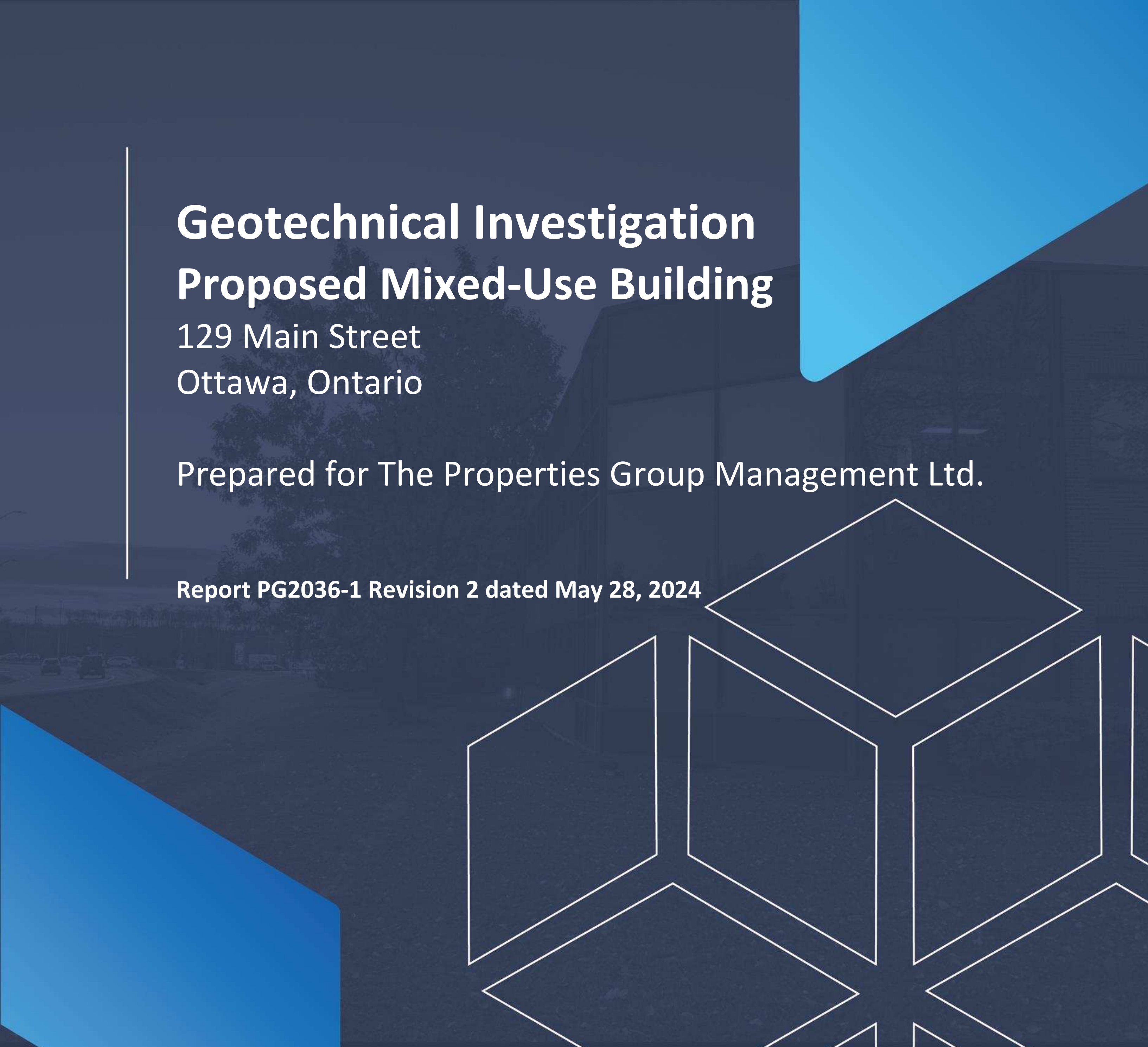

Geotechnical Investigation
Proposed Mixed-Use Building

129 Main Street
Ottawa, Ontario

Prepared for The Properties Group Management Ltd.

Report PG2036-1 Revision 2 dated May 28, 2024

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by The Properties Group Management Ltd. 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 geotechnical investigation were to:

- ☐ Determine the subsoil and groundwater conditions at this site by means of test holes.

- ☐ Provide geotechnical recommendations pertaining to 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 for the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

It is understood that the proposed project will consist of a six (6) storey mixed-use apartment building with two underground parking levels. It is further understood that the proposed building footprint will take up the majority of the subject site. Associated landscaped and hardscaped areas are also anticipated as part of the development. It is further understood that the subject site is municipally serviced.

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 the 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 PG2036-2 - Test Hole Location Plan presented 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 Tests (SPT) were 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 the location of borehole 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. 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 attached to the current report.

Groundwater

Flexible standpipes were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. In addition, groundwater observations were recorded in the open hole test pits during the current geotechnical investigation.

The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

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 PG2036-2 - 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 property is vacant, and gravel-covered. The subject site is bordered by two two-storey buildings 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 neighboring building is founded along the subject site's north property boundary. The ground surface slopes gradually down towards the south across the site from 65.0 m to 64.5 m.

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

Overburden

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.

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.

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of shale of the Billings Formation. with an approximate overburden thickness across the majority of the subject site ranging between 25 to 50 m.

4.3 Groundwater

Groundwater levels (GWLs) were measured in the standpipes installed in the boreholes and the results are summarized in Table 1.

Table 1 – Summary of Groundwater Levels

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

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.

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations.

The Long-term groundwater levels can also be estimated based on the observed color, consistency, and moisture content of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at an approximate depth of **2.5 to 3.5 m** below the ground surface. Groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed building. With two (2) levels of underground parking, the founding elevation is approximately 7 m below the 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, asphalt, and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding, and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

Fill Placement

Fill placed for grading throughout the building footprint should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. Imported fill material should be tested and approved prior to delivery to the site. The fill should be placed in a maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. These materials should be spread in a maximum of 300 mm thick loose lifts and compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in maximum 300 mm thick loose lifts to at least 98% of the material's SPMDD.

The placement of subgrade material should be reviewed at the time of placement by Paterson personnel. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Terraxx.

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. This material should be tested and approved by Paterson 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 paved areas should be compacted to at least 100% of its SPMDD.

5.3 Foundation Design

Bearing Resistance Value (Conventional Shallow Foundation)

Footings, up to 6 m wide, founded on an 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.

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.

Permissible Grade Raise Recommendations

Based on the undrained shear strength values of the silty clay deposit encountered throughout the subject site and our experience with the local silty clay deposit, a **permissible grade raise restriction of 1.5 m** is recommended in the immediate area of settlement sensitive structures. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise restriction calculations.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

Settlement

Footings bearing on an undisturbed soil bearing surface and designed using the bearing resistance values provided above 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 the in-situ bearing medium soils above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

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 **200 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 **300 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 **8 MPa/m** for a contact pressure of **200 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.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012.

The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 2 and 3 in Appendix 2 of the present report. It should be noted that the shear wave velocity calculations take into consideration an assumed underside of footing depth of 7 m.

Field Program

The seismic array was located within the proposed building footprint, as presented in Drawing PG2036-2 - Test Hole Location Plan attached to the present report. Paterson field personnel placed 20 horizontal geophones in a straight line in roughly a northwest-southeast 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 2.0 and 3.0 m away from the first geophone, 2.0, 3.0 and 22.0 m away from the last geophone, and at the centre of the geophone array.

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 performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile immediately below the building 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, the bedrock surface is located at 26.5 m below the existing ground surface. However, the bedrock velocity could not be accurately defined. Therefore, 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.

For conventional footings placed on the overburden, with an assumed underside footing depth of 7 m below existing ground surface, the Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012.

$$V_{s30} = \frac{Depth_{of\ interest}(m)}{\left(\frac{Depth_{Layer1}(m)}{V_{s_{Layer1}}(m/s)} + \frac{Depth_{Layer2}(m)}{V_{S_{Layer2}}(m/s)}\right)}$$

$$V_{s30=} \frac{30\ m}{\left(\frac{(26.5 - 7.0)\ m}{197\ m/s} + \frac{10.5\ m}{1500\ m/s}\right)}$$

$$V_{s30=}\ 283\ m/s$$

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 **283 m/s**. Therefore, a **Site Class D** is applicable for the 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 understood that the basement walls are to be poured against a water suppression system, which will be placed 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, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

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

Lateral Earth Pressures

The static horizontal earth pressure (Po) 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. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

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

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area, is 0.32 g according to the OBC 2012. Note that the vertical seismic coefficient is assumed to be zero. The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

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

5.7 Pavement Design

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 in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil.	

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 in situ soil, fill 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 II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, such as Terratrack 200 or equivalent, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage and Waterproofing

The following recommendations may be considered for the architectural design of the building's foundation drainage systems. It is recommended that Paterson be engaged at the design stage of the future building (and prior to tender) to review and provide supplemental information for the building foundation drainage system design.

Supplemental details, review of architectural design drawings, and additional information may be provided by Paterson for these items for incorporation in the building design packages and associated tender documents. It is recommended that Paterson review all details associated with the foundation drainage system prior to tender.

Groundwater Suppression System

It is recommended that a groundwater suppression system be provided for the proposed structure. It is expected that insufficient room will be available for exterior backfill and the foundation wall will be cast as a blind-sided pour against a shoring system and the bedrock surface. It is recommended that the groundwater suppression system consist of the following:

- ☐ A waterproofing membrane should be placed against the shoring system between the underside of footings and 1 m below the existing ground surface (1 m above long-term groundwater elevation). Where the membrane will extend against the shoring system, it is recommended to consist of a membrane with a bentonite-lined face for being paced against the shoring system. The membrane is recommended to overlap below the overlying perimeter foundation footprint by a minimum of 1 m inwards towards the building footprint and from the face of the overlying foundation. This will allow construction to proceed without imposing groundwater lowering within the surrounding area of the proposed building in the short and long term conditions.

- ☐ A composite drainage membrane (Delta Terraxx, MiraDrain G100N or equivalent) should be placed against the HDPE face of the waterproofing membrane with the geotextile layer facing the waterproofing layer from finished ground surface to the top of the footing.

- ☐ The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by the geotechnical consultant. It is highly recommended that the drainage board rolls be installed horizontally rather than vertically to minimize the number of vertical joints forming between the rolls.

- ☐ It is recommended that 150 mm diameter PVC sleeves at 6 m centers be cast in the foundation wall at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The sleeves should be connected to openings in the HDPE face of the drainage board layer. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area via an underfloor and interior drainage pipe system.

The top end lap of the foundation drainage board should be provided with a suitable termination bar against the foundation wall to mitigate the potential for water to perch between the drainage board and foundation wall.

Interior Perimeter and Underfloor Drainage

An interior perimeter and underfloor drainage system will be required to redirect water from the building’s foundation drainage system to the building’s sump pit(s) if it will not discharge to an exterior catch basin structure. For preliminary design purposes, it is recommended that the interior perimeter and underfloor drainage pipes should consist of 100 or 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock, placed at approximately 6 m.

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe. The interior drainage pipe should be provided tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves.

The spacing of the underfloor drainage should be confirmed by Paterson at the time of excavation when water infiltration can be better assessed and once the foundation layout and sump system location has been finalized.

Foundation Backfill

Where applicable, 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 drainage geocomposite, such as Miradrain G100N or Delta Terraxx, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type II granular material, should otherwise be used for this purpose.

Foundation backfill material should be compacted in maximum 300 mm thick loose lifts and with suitably sized vibratory compaction equipment (smooth-drum roller for crushed stone fill, sheepsfoot roller for soil fill).

Podium Deck Waterproofing Tie-In (If Applicable)

Waterproofing layers for podium deck surfaces should overlap across and below the top end lap of the vertically installed composite foundation drainage board to mitigate the potential for water to migrate between the drainage board and foundation wall as depicted in Figure 4 – Podium Deck to Foundation Wall Drainage System Tie-in Detail.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

Foundation Raft Slab Construction Joints

If applicable, it is anticipated the raft slab will be poured in several pour segments. For the construction joint at each pour, a PVC water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab. Furthermore, a PVC water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Finalized Drainage and Waterproofing Design

Paterson should be provided with the finalized structural and architectural drawings for the proposed building to provide a building-specific waterproofing and drainage design which includes the above-noted recommendations. The design will provide recommendations for other items such as minimum pipe spacings, pipe mechanical connections below grade, transitioning from blind to double sided pours (if applicable), etc.

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 1.5 m thick soil cover alone, or a combination of soil cover in conjunction with foundation insulation should be provided in this regard.

Other 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 proper structure. These footings should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

6.3 Excavation Side Slopes

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.

Unsupported Side Slopes

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 subsurface soil is considered to be mainly a Type 2 and Type 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 maintain safe working distance from the excavation sides.

Excavation side slopes carried out for the building footprint are recommended to be provided surface protection from erosion by rain and surface water runoff where shoring is not anticipated to be implemented. This can be accomplished by covering the entire surface of the excavation side-slopes with tarps secured between the top and bottom of the excavation and approved by Paterson personnel at the time of construction.

It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of side-slopes.

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.
A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain 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 for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
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
Submerged 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.

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular A. The bedding layer thickness should be increased to a minimum of 300 mm where the subgrade will consist of grey silty clay or bedrock. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

It should generally be possible to re-use the moist (not wet) site-generated fill above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet site-generated fill, such as the grey silty sand, will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

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

6.5 Groundwater Control

Groundwater Control for Building Construction

Based on our observations, 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. Provisions should be carried out for using higher capacity open sump systems for excavations undertaken below the bedrock surface. The contractor should be prepared to direct water away from all subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

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

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

6.6 Winter Construction

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

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

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

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

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.

6.8 Landscaping Considerations

Tree Planting Considerations

It is understood the proposed buildings will include two to six levels of underground parking and the structures will be founded on foundation located at a minimum of 7 m below finished grade. Given the depth of foundations proposed for the structures, it is expected that the support of the foundations derives from soil located below the depth that dewatering by tree roots. Therefore, foundation distress due to potential moisture depletion caused by trees is not expected to occur at the subject site. Since the structures are not anticipated to be founded upon silty clay soils affected by the depth of root penetration, City approved trees within the subject site will not be subject to planting restrictions as based on the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines) from a geotechnical perspective.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- ☐ Review preliminary and detailed grading, servicing, landscaping, and structural plan(s) from a geotechnical perspective.
- ☐ Review of the geotechnical aspects of the excavation contractor's shoring design, if not designed by Paterson, prior to construction, if applicable.
- ☐ Review of architectural plans pertaining to groundwater suppression systems, underfloor drainage systems, and waterproofing details for elevator shafts.

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

- ☐ Review and inspection of the installation of the foundation drainage systems.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Observation of driving and re-striking of all pile foundations.
- ☐ Sampling and testing of the concrete and fill materials.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per ***Ontario Regulation 406/19: On-Site and Excess Soil Management*.**

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with the present understanding of the project. Paterson requests permission to review the 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, Paterson requests 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 or 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 The Properties Group Management Ltd., 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.

Yashar Ziaeimehr, M.A.Sc.

LICENSED PROFESSIONAL ENGINEER
May 28, 2024
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ☐ The Properties Group Management Ltd. (Email Copy)
- ☐ 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

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

○ Water Content %
20 40 60 80

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
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			3	61.89	▽
End of Test Pit 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)								

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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, trace construction debris — 0.41		G	3				
		G	4			1	63.49
FILL: Brown silty sand, trace gravel							
		G	5			2	62.49
— 3.00		G	6				
Stiff, grey **SILTY CLAY,** some sand		G	7			3	61.49
— 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

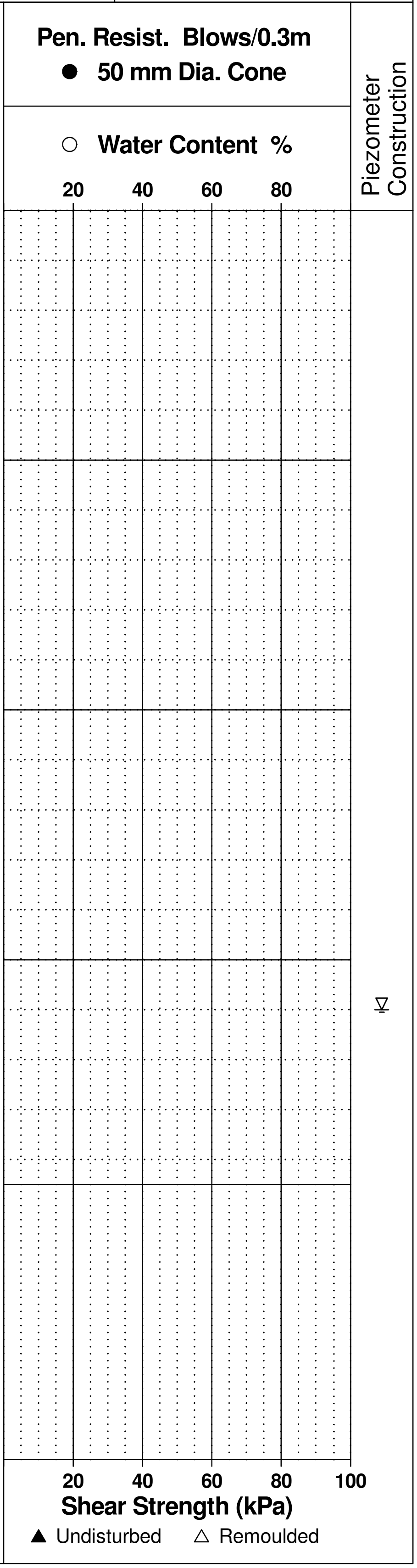
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.71
FILL: Crushed stone with silt, trace organics 0.23		G	1				
FILL: Brown silty sand, trace gravel		G	2			1	63.71
		G	3				
1.80		G	4				
						2	62.71
FILL: Grey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

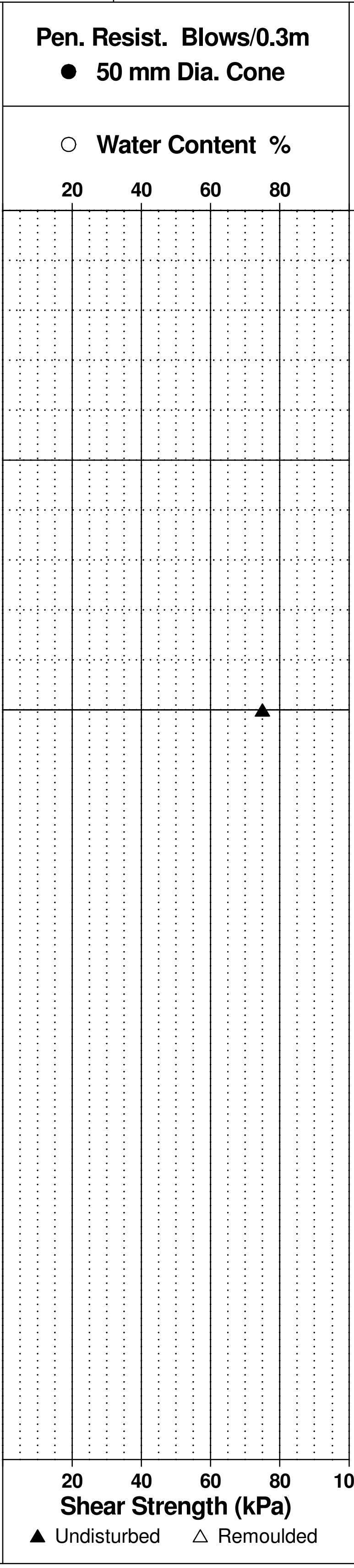
BORINGS BY Backhoe

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)							



Piezometer Construction

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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

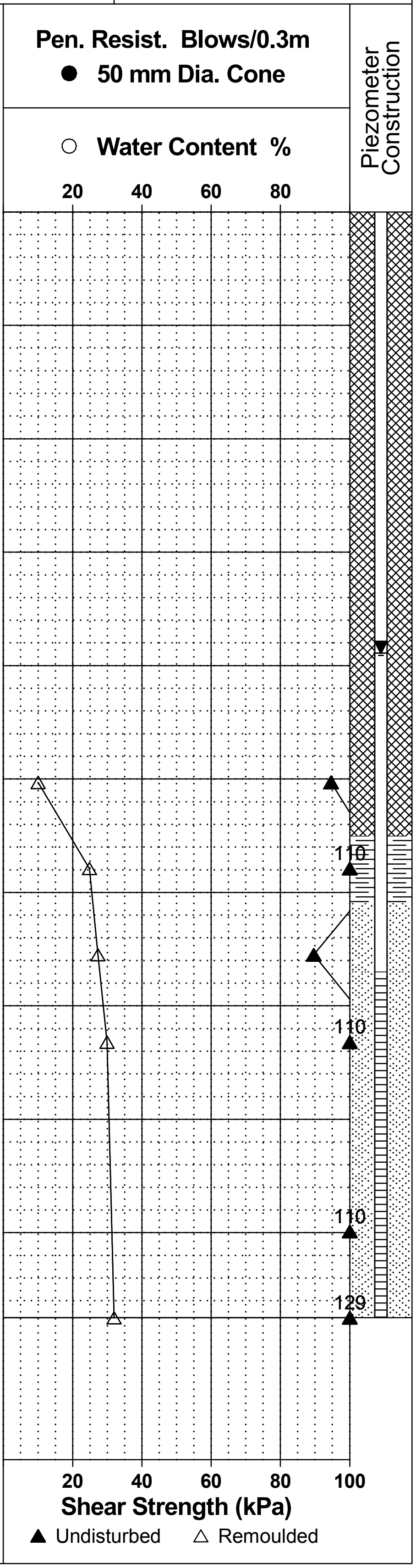
BORINGS BY CME 75 Power Auger

DATE March 22, 2010

FILE NO. PG2036

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
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		
Stiff to very stiff, dark grey **SILTY CLAY** with black organic mottling		SS	6	100	2	4	60.42
						5	59.42
						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)							



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

GROUND SURFACE

FILL: Grey silty sand and gravel — 0.15

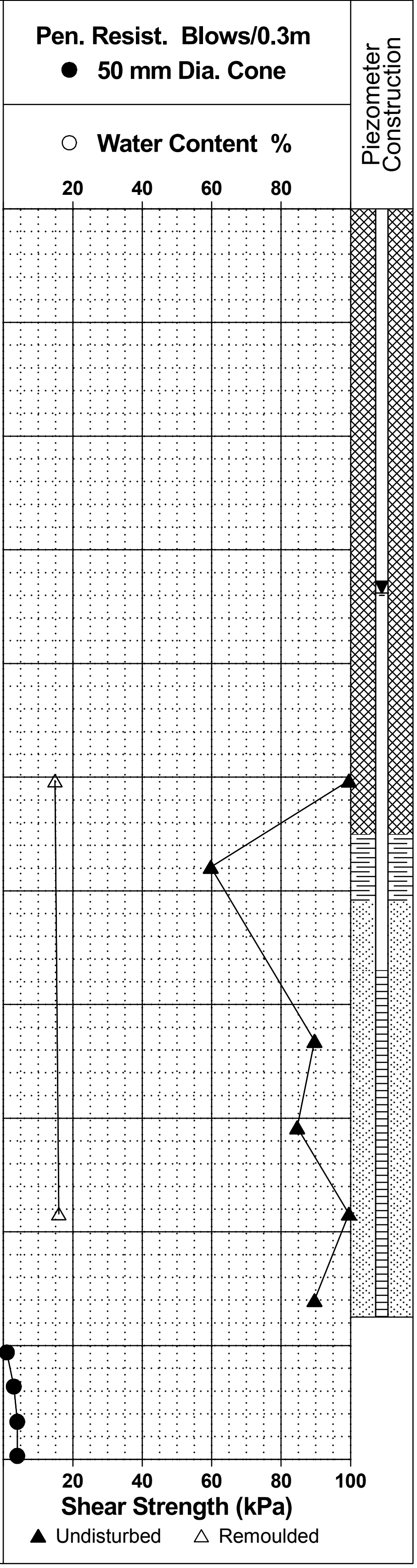
FILL: Brown silty sand, trace gravel — 3.66

Stiff, dark grey **SILTY CLAY** — 9.75

Dynamic Cone Penetration Test commenced @ 9.75m depth

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	31
SS	3	50	20
SS	4	58	10
SS	5	25	7
SS	6	100	2
SS	7	100	2
SS	8	100	

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



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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

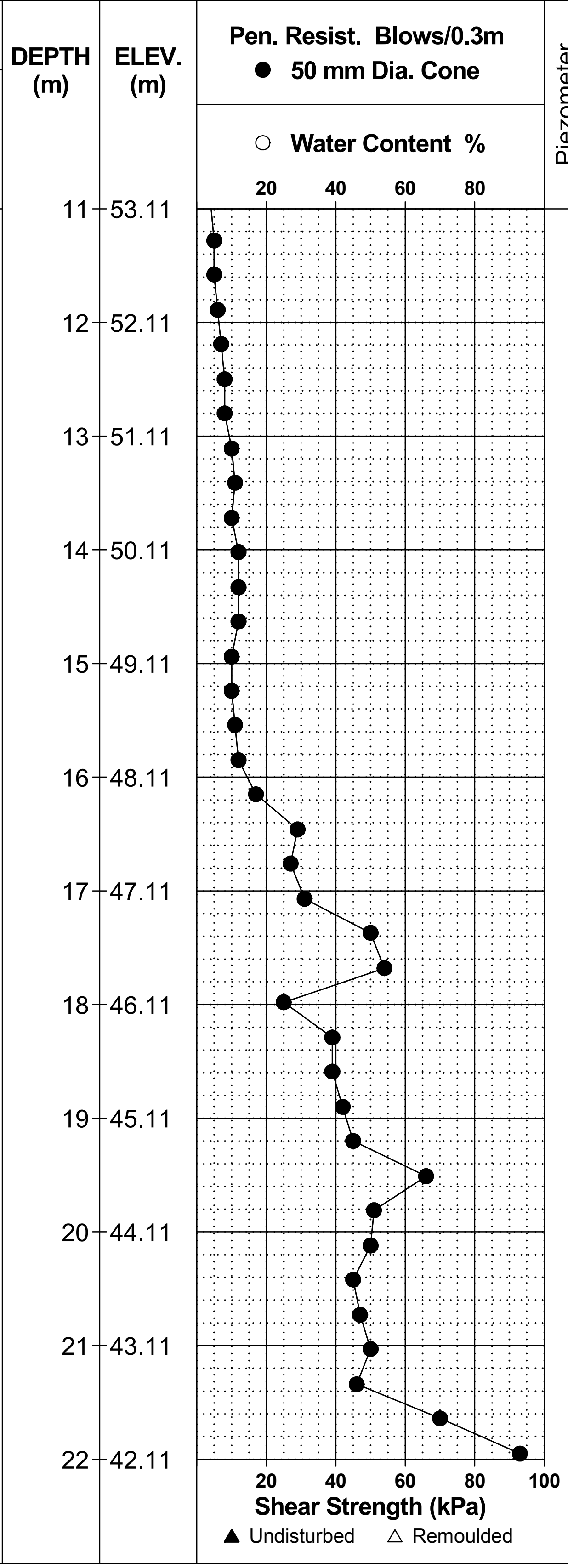
BORINGS BY CME 75 Power Auger

DATE March 22, 2010

FILE NO. PG2036

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	Piezometer Construction
GROUND SURFACE						



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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

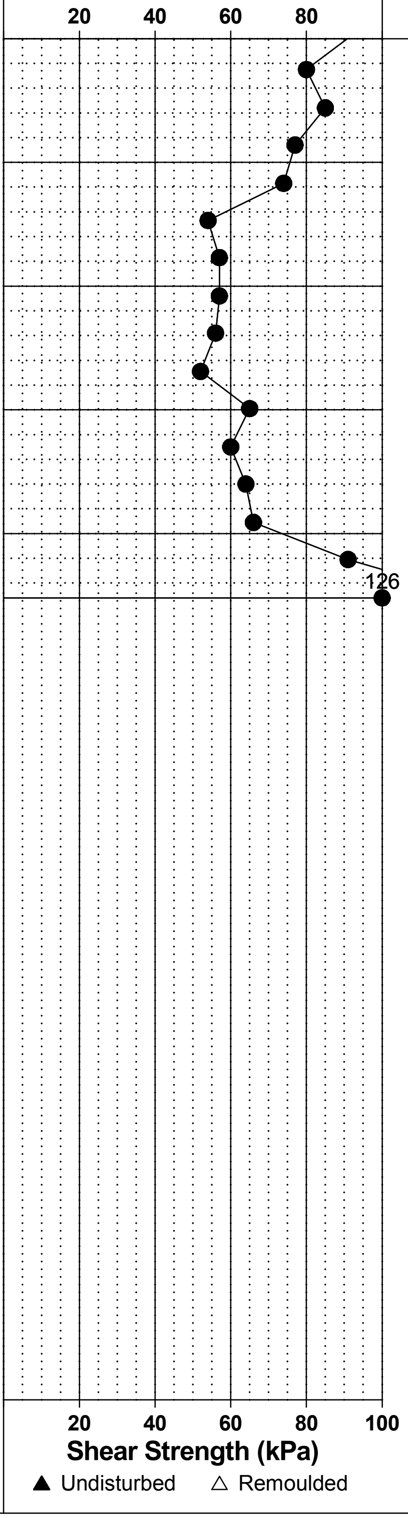
BORINGS BY CME 75 Power Auger

DATE March 22, 2010

FILE NO. PG2036

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE									
						22	42.11		
						23	41.11		
						24	40.11		
						25	39.11		
						26	38.11		
26.52									
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

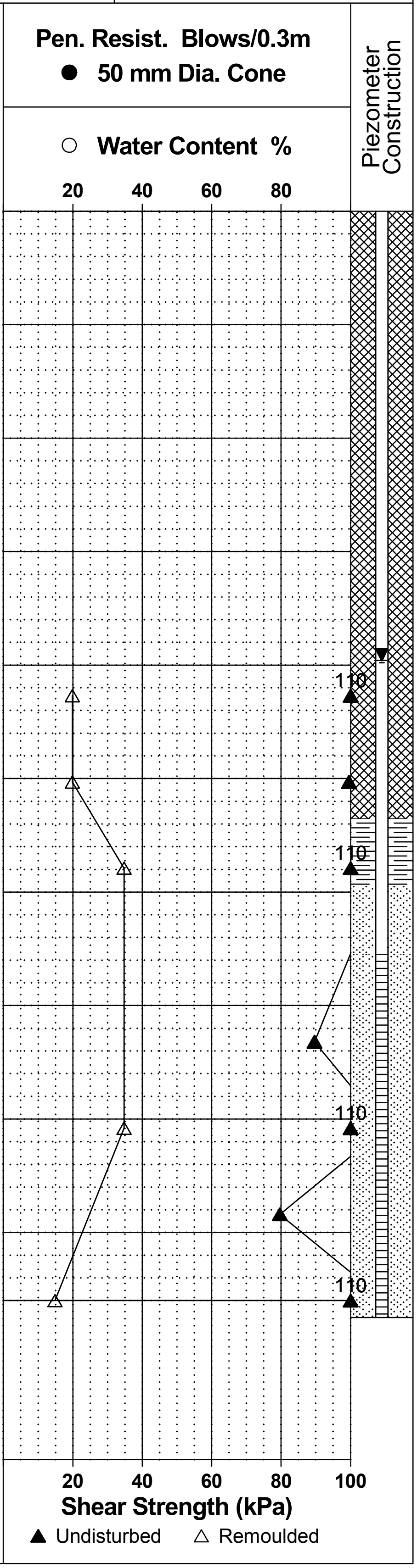
FILE NO. PG2036

HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
FILL: Grey silty sand and gravel	0.15
FILL: Brown silty sand, trace gravel	2.90
Very stiff to stiff, dark grey **SILTY CLAY**	9.60
End of Borehole	
(GWL @ 3.96m-Mar. 25/10)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	46	9
SS	3	38	9
SS	4	21	17
SS	5	83	3
SS	6	100	3

DEPTH (m)	ELEV. (m)
0	64.86
1	63.86
2	62.86
3	61.86
4	60.86
5	59.86
6	58.86
7	57.86
8	56.86
9	55.86



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 = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = $D60 / D10$

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

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

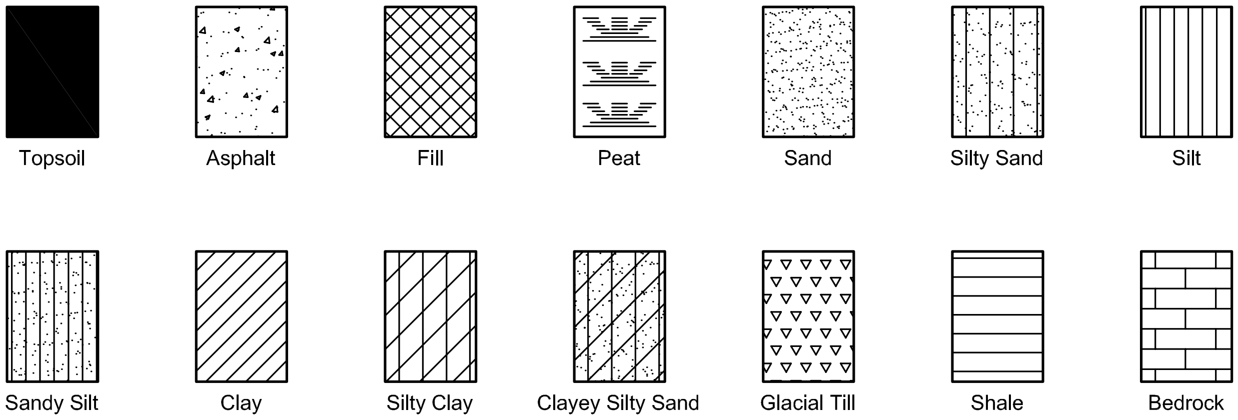
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

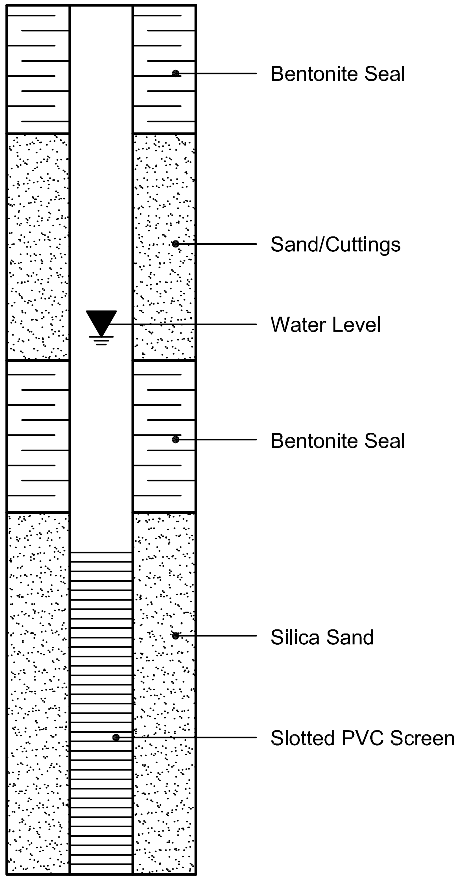
SYMBOLS AND TERMS (continued)

STRATA PLOT

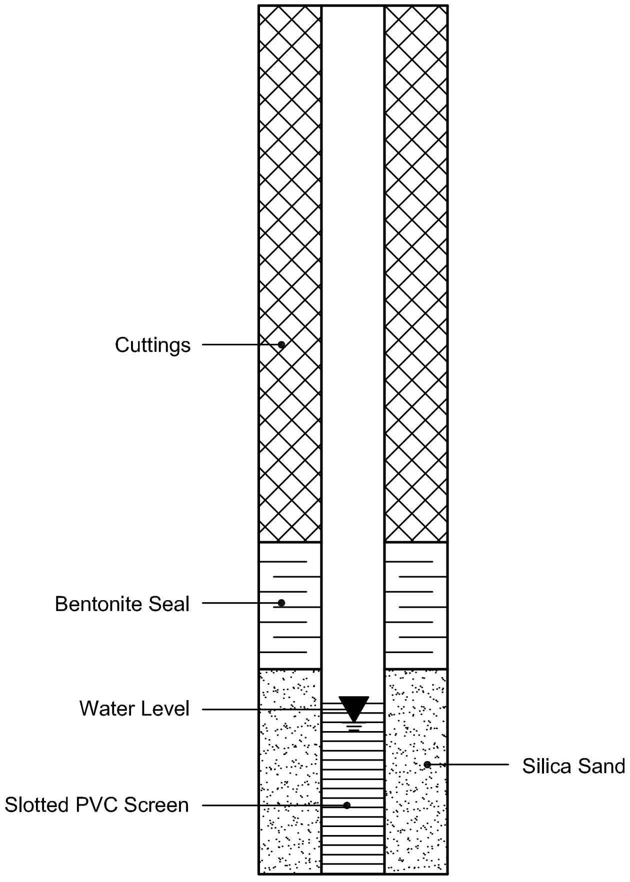


MONITORING WELL AND PIEZOMETER CONSTRUCTION

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

FIGURES 2 AND 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURE 4 – PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

DRAWING PG2036-2 - TEST HOLE LOCATION PLAN

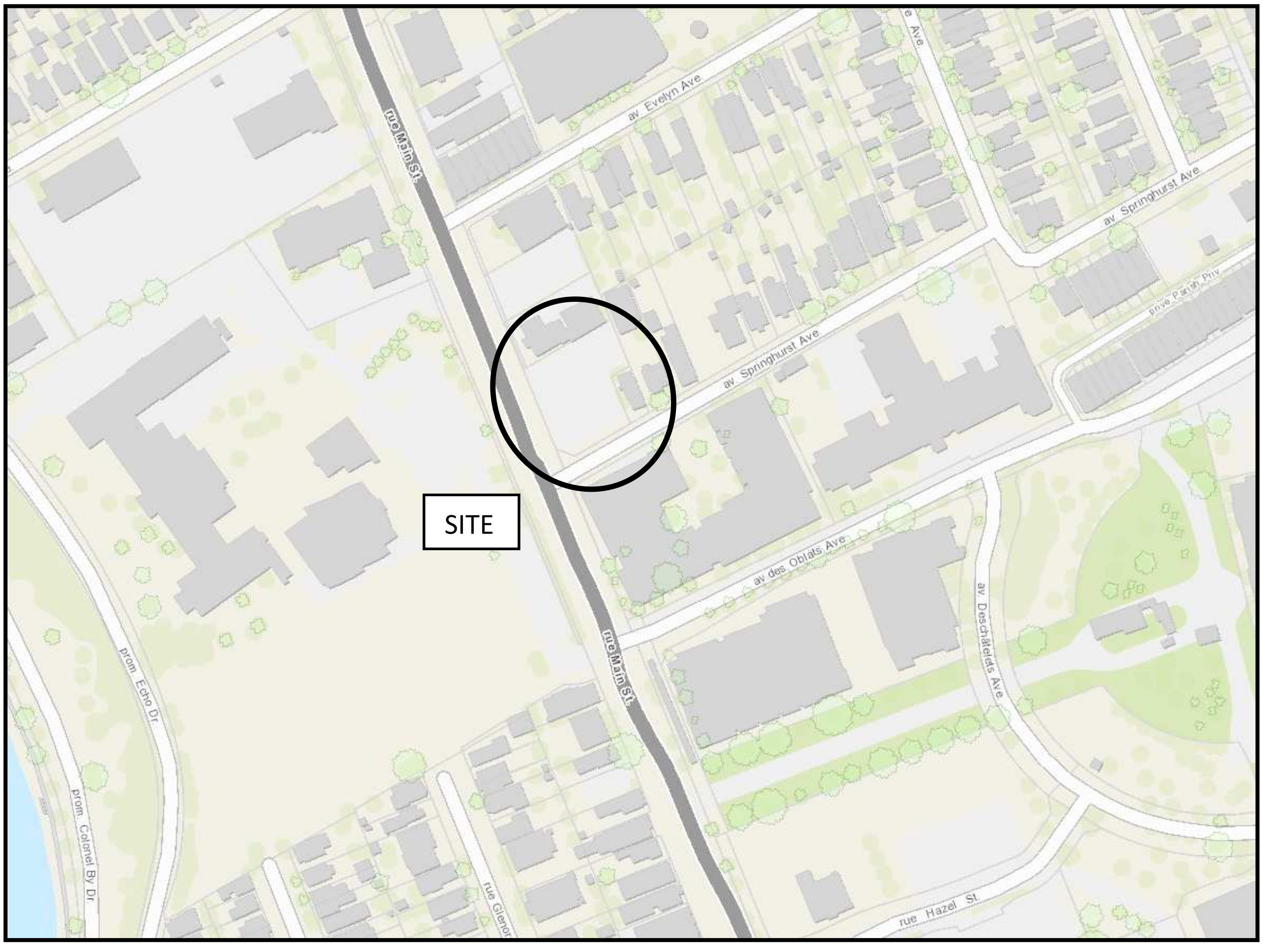


FIGURE 1

KEY PLAN

PATERSON GROUP

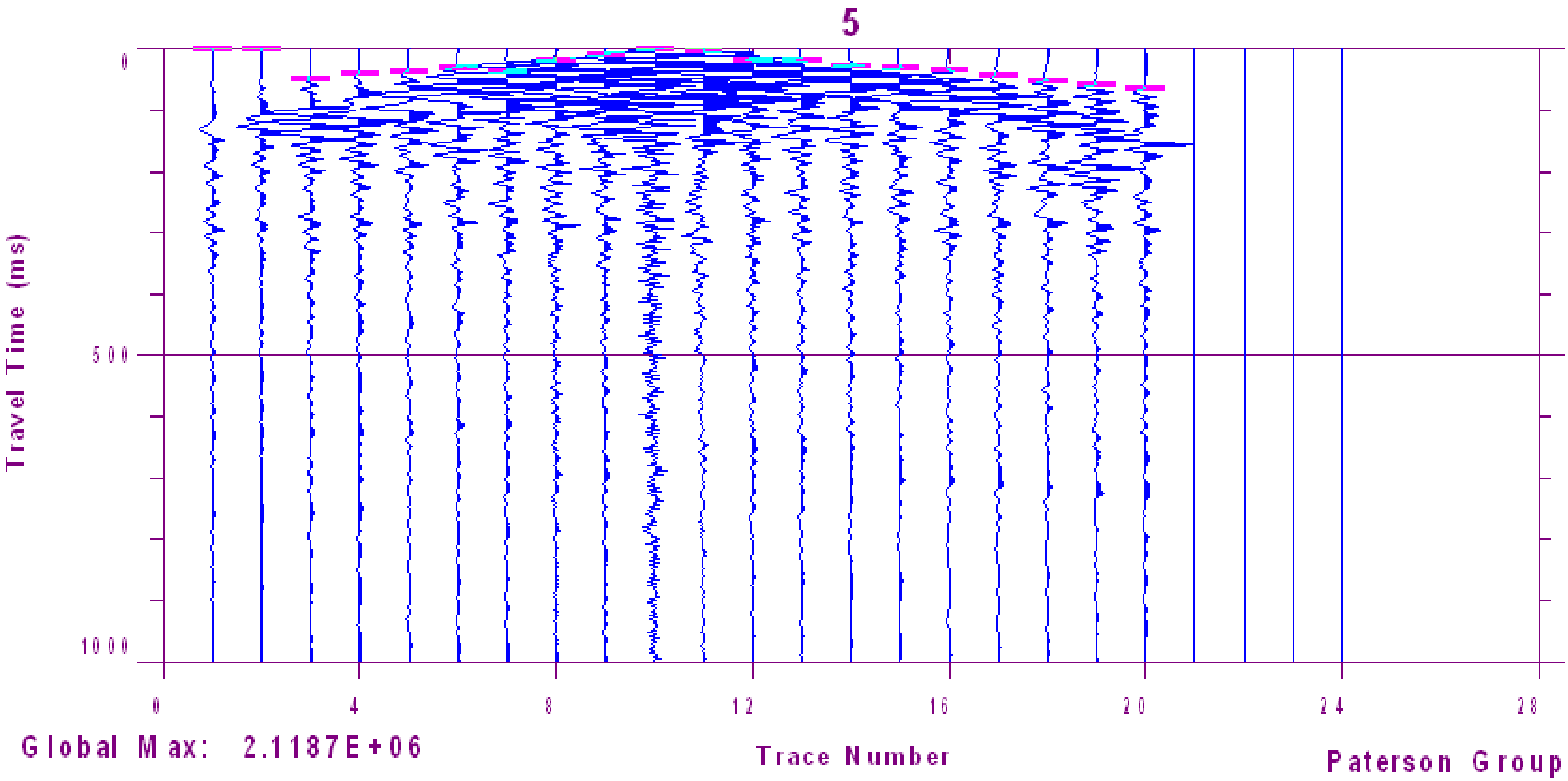


Figure 2 – Shear Wave Profile at Shot Location 19.0 m

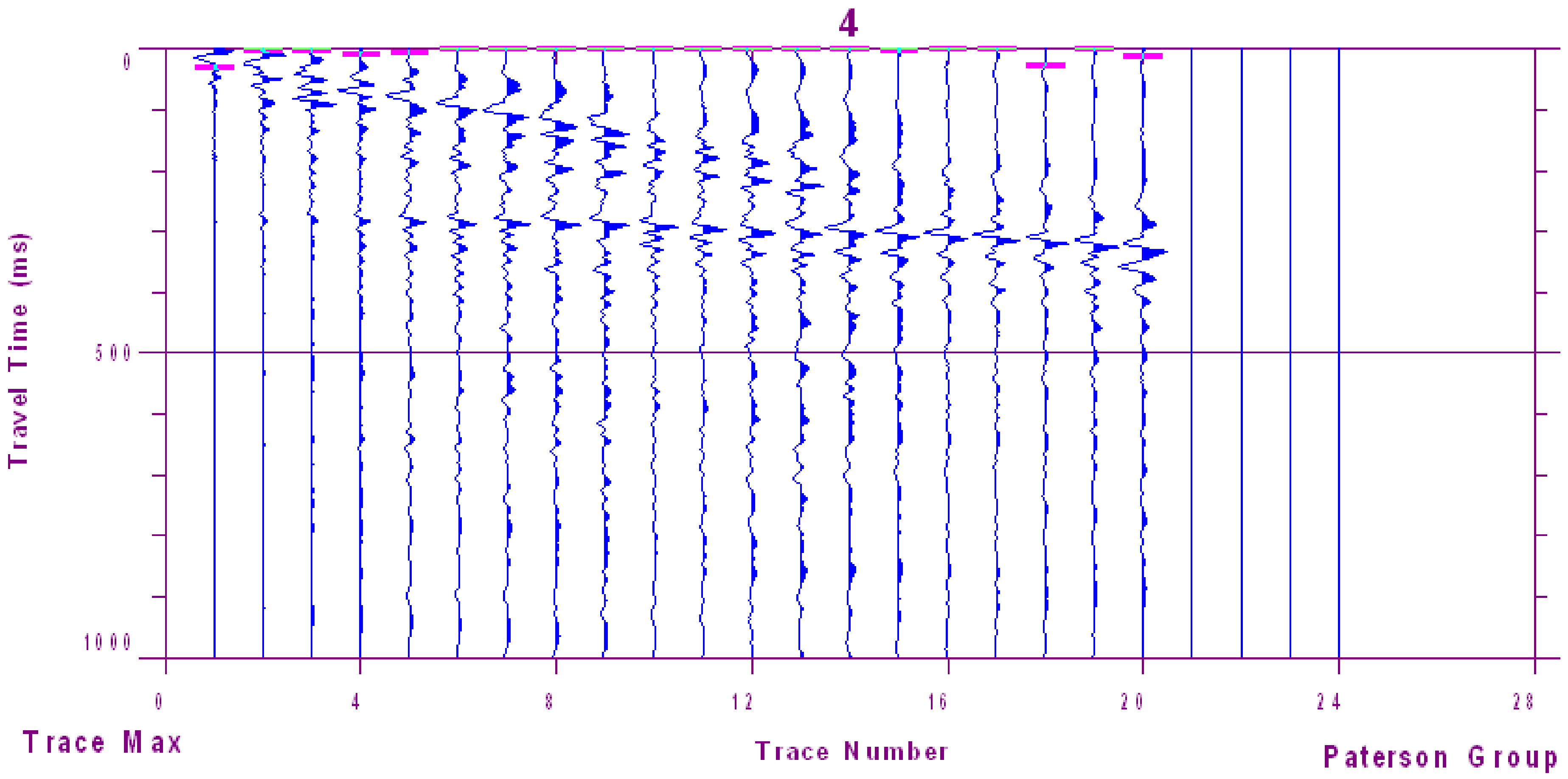
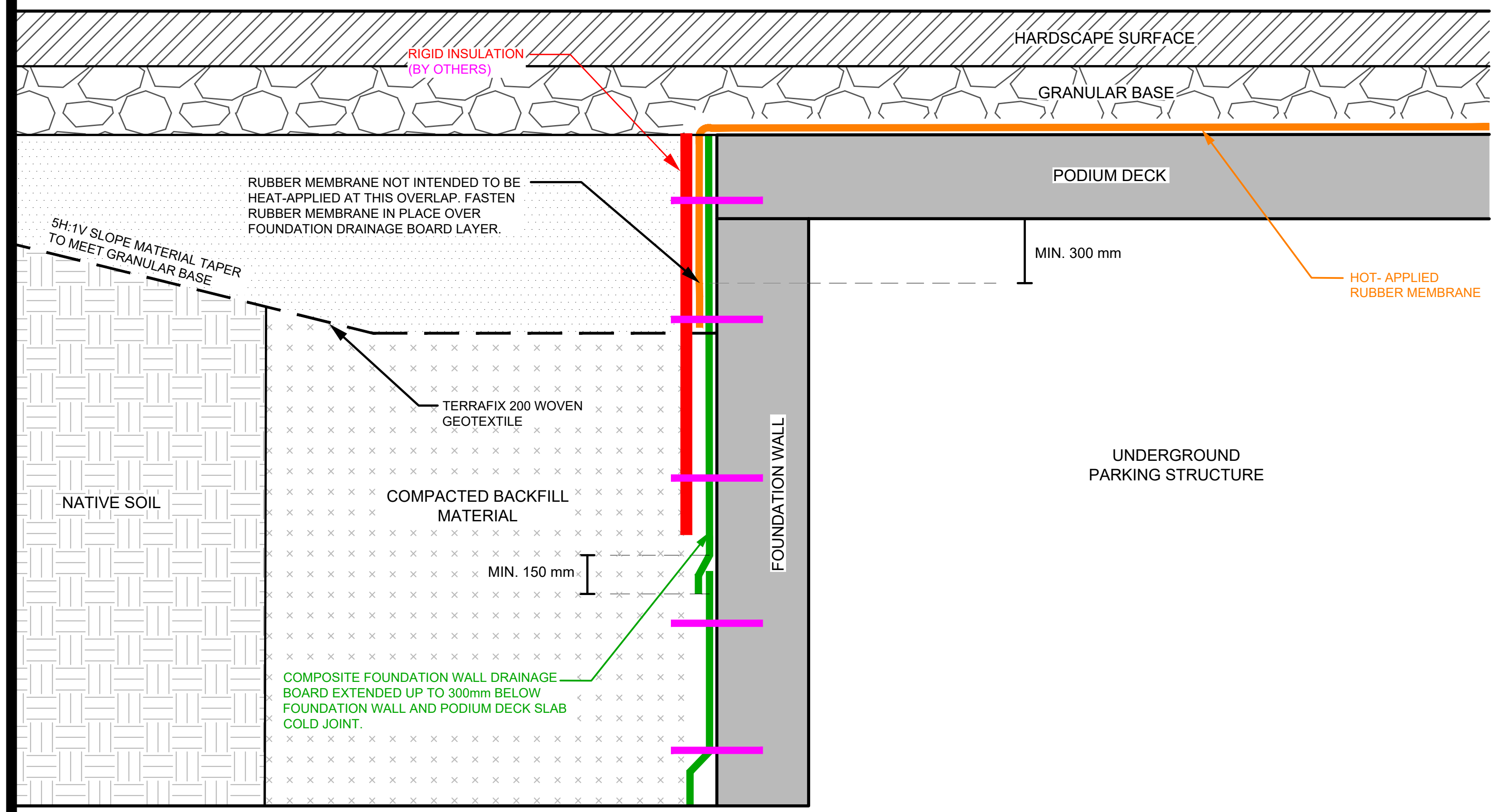
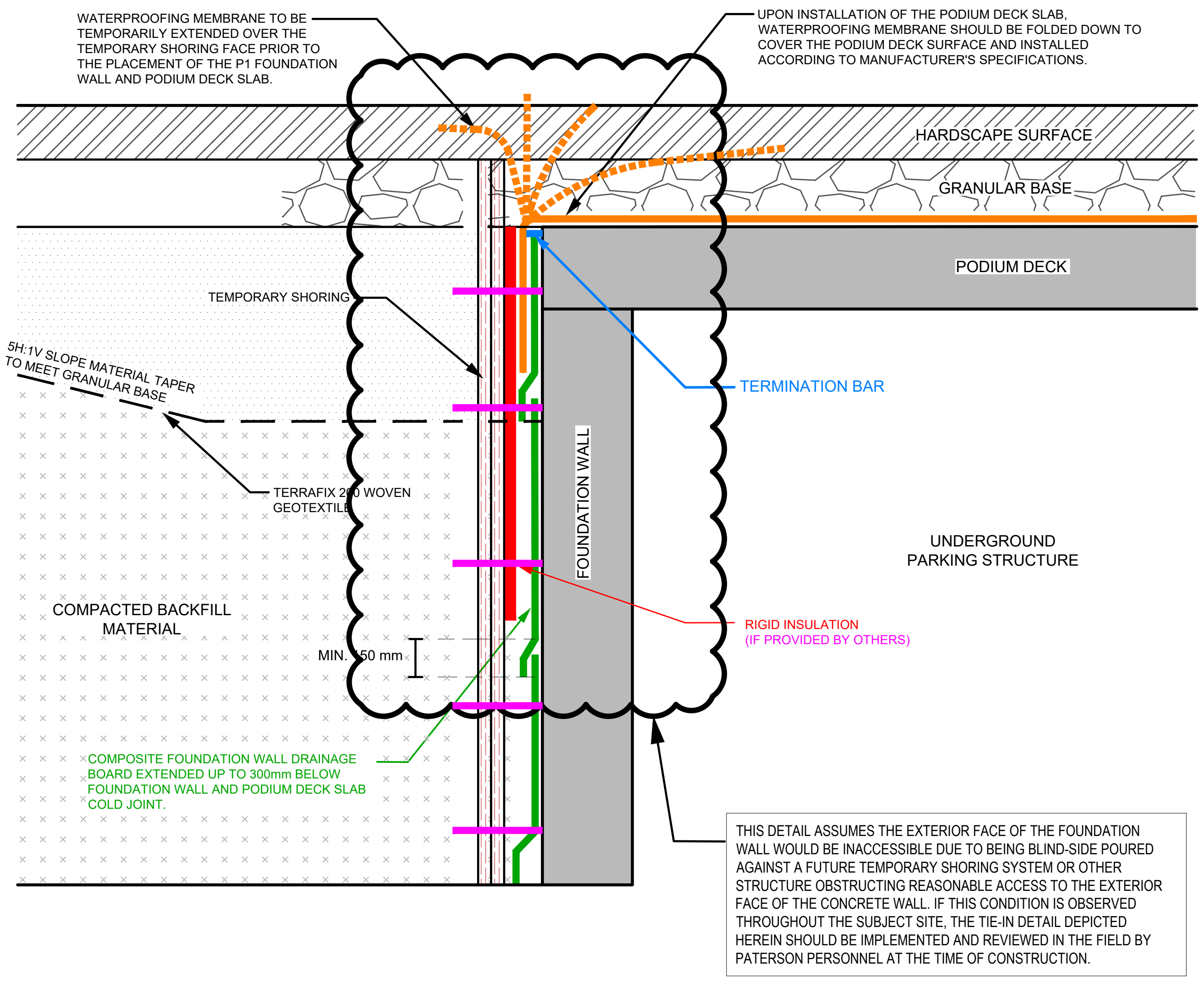


Figure 3 – Shear Wave Profile at Shot Location -2.0 m

OPTION A - DOUBLE-SIDE POURED TOP OF FOUNDATION WALL



OPTION B - BLIND-SIDE POURED TOP OF FOUNDATION WALL



THIS DETAIL ASSUMES THE EXTERIOR FACE OF THE FOUNDATION WALL WOULD BE INACCESSIBLE DUE TO BEING BLIND-SIDE POURED AGAINST A FUTURE TEMPORARY SHORING SYSTEM OR OTHER STRUCTURE OBSTRUCTING REASONABLE ACCESS TO THE EXTERIOR FACE OF THE CONCRETE WALL. IF THIS CONDITION IS OBSERVED THROUGHOUT THE SUBJECT SITE, THE TIE-IN DETAIL DEPICTED HEREIN SHOULD BE IMPLEMENTED AND REVIEWED IN THE FIELD BY PATERSON PERSONNEL AT THE TIME OF CONSTRUCTION.

NOTES:

THE ABOVE DETAIL FOR HOT RUBBER AND DRAINAGE BOARD OVERLAP IS APPLICABLE TO ALL EDGE-PORTIONS OF THE PODIUM DECK AND/OR SUSPENDED GROUND FLOOR SLAB STRUCTURE.

APPLICABILITY THICKNESS AND EXTENSIONS OF RIGID INSULATION ARE SPECIFIED BY OTHERS

WHERE THE GRADING SURFACE TERMINATES AGAINST THE BUILDING FACE AND PAVEMENT STRUCTURE IS NOT LOCATED ABOVE THE EDGE OF THE FOUNDATION WALL AND PODIUM DECK SLAB AS DEPICTED HEREIN, IT IS RECOMMENDED TO PROVIDE A SUITABLE TERMINATION BAR TO SEAL THE TOP ENDLAP OF THE HOT-APPLIED RUBBER MEMBRANE LAYER TO THE VERTICAL FACE OF THE STRUCTURE. THIS WOULD BE REQUIRED TO MITIGATE THE POTENTIAL FOR THE MIGRATION OF WATER BEHIND THE RUBBER MEMBRANE.

ALL PORTIONS OF THE ABOVE-NOTED DETAIL (INSULATION OF FOUNDATION DRAINAGE BOARD, TERMINATION BAR, HOT-RUBBER MEMBRANE OVER SLAB, FOUNDATION WALL CONSTRUCTION JOINT AND OVERLAPPING/SHINGLING OF DRAINAGE BOARD) SHOULD BE REVIEWED AT THE TIME OF CONSTRUCTION BY PATERSON PERSONNEL.

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
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NO.	REVISIONS	DATE	INITIAL

THE PROPERTIES GROUP MANAGEMENT LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE BUILDING
129 MAIN STREET

OTTAWA, ONTARIO

Title:
PODIUM DECK TO FOUNDATION WALL DRAINAGE SYSTEM TIE-IN DETAIL

Scale: N.T.S
Drawn by: ZS
Checked by: NP
Approved by: FA
Date: 05/2024
Report No.: PG2036
Dwg. No.: **FIGURE 4**
Revision No.:

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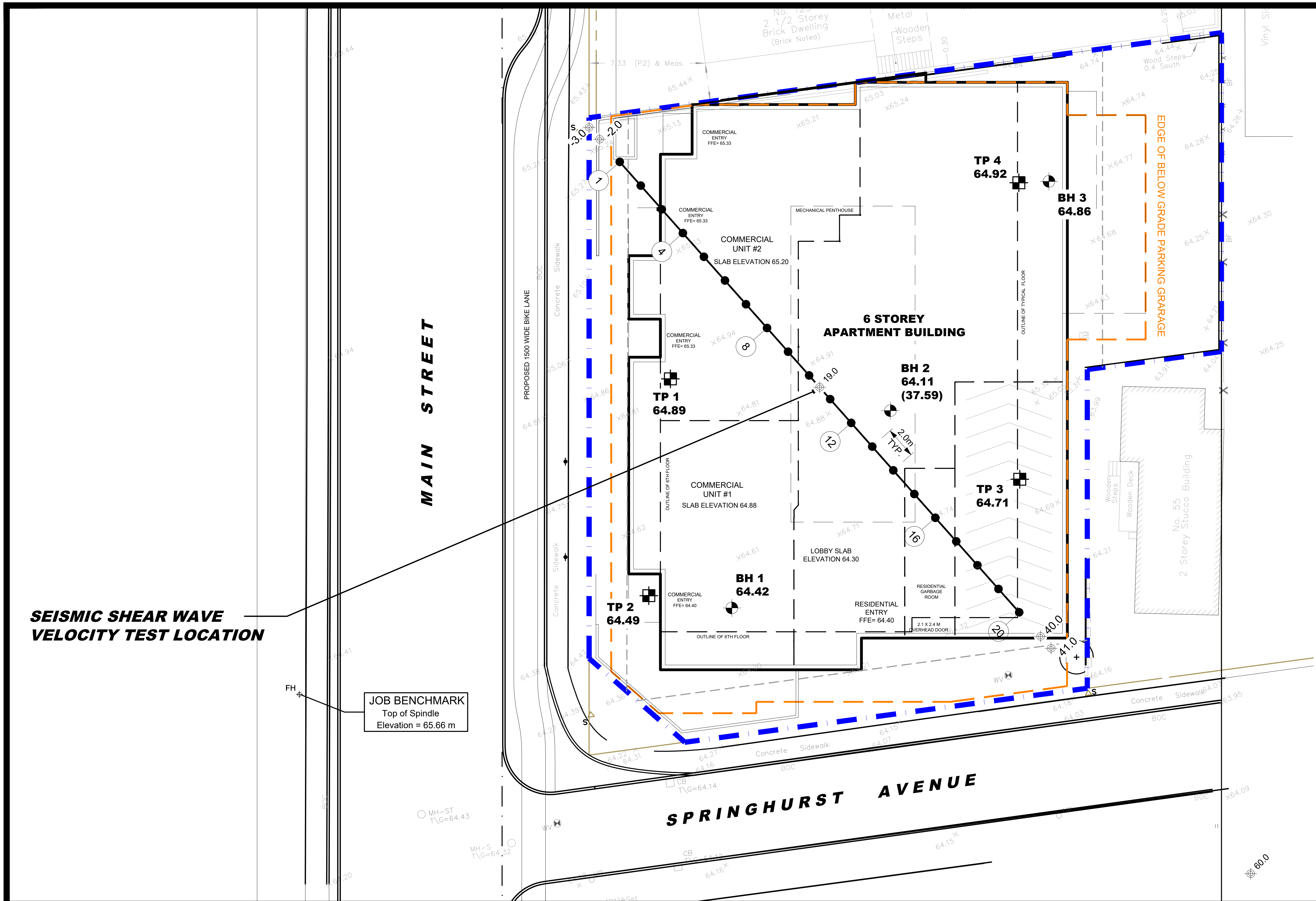

LEGEND:

▣	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

CONCEPTUAL PLAN PROVIDED BY RODERICKLAHEY ARCHITECT INC.

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

SCALE: 1:250

0 1 2 3 4 5 10 15m

PATERSON GROUP

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NO.	REVISIONS	DATE	INITIAL
2	CONCEPTUAL PLAN UPDATED	30/04/2024	YZ
1	BASE PLAN UPDATED	25/06/2018	DJG

THE PROPERTIES GROUP LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE BUILDING - 129 MAIN STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250
Date: 07/2017
Drawn by: GK
Report No.: PG2036-1
Checked by: NP
Dwg. No.: PG2036-2
Approved by: FA
Revision No.: 2

11x17

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