PATERSON GROUP

Geotechnical Investigation

Proposed Residential Development

114 Richmond Road
Ottawa, Ontario

Prepared for Concorde Properties

Report PG7871-1 dated February 24, 2026

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Concorde Properties to prepare a Geotechnical Investigation Report for the proposed residential development to be located at 114 Richmond Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 for the general site location).

The objective of the geotechnical investigation was to:

- ❑ Determine the subsoil and groundwater conditions at this site by means of existing 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 the presence or potential presence of contamination on the subject site 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

Based on the available drawings, it is understood that the proposed development will consist of back-to-back and stacked townhouses either with partial basements or overlying one level of underground parking.

A four-storey residential building with one level of underground parking level, is proposed at the northern portion of the site. Demolition of a portion of the existing building will be required for the development of this portion of the site.

Around the proposed buildings, the remainder of the site will generally consist of asphalt-paved access lanes and parking areas with landscaped margins.

It is also expected that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

Previous investigations were carried out by Paterson at the subject site in May 2023, February 2021 and in July 2010. A total of eight (8) boreholes and five (5) test pits were advanced to maximum depths of 15.0 and 4.0 m during the previous investigations. The test hole locations are shown on Drawing PG7871-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a track-mounted drill rig operated by a two-person crew. The test pits were excavated using a hydraulic shovel and backfilled with the excavated soil upon completion. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The test hole procedure consisted of augering or excavating and bedrock coring to the required depths at the selected locations, and sampling and testing the overburden and bedrock.

Sampling and In-Situ Testing

Soil samples were recovered from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using a 47.6 mm inside diameter coring equipment. Soil samples were also recovered from the sidewalls of the test pits (G). All soil samples were visually inspected and classified on site. The auger, split spoon, and grab samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification.

The depths at which the auger, split-spoon, grab and rock core samples were recovered from the boreholes are shown as AU, SS, G and RC, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples and 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.

Bedrock samples were recovered from boreholes BH 1-21 through BH 3-21 and BH 2 through BH 4 using a core barrel and diamond drilling techniques. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. The values are indicative of the bedrock quality.

The 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

Groundwater monitoring wells were installed in boreholes BH 1-21, BH 2-21 and BH 3-21 during the geotechnical investigation to permit monitoring of groundwater levels subsequent to the completion of the sampling program. Flexible piezometers were installed in boreholes BH 1 through BH 5. The groundwater observations are discussed in Section 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The location and ground surface elevation at each borehole completed during the February 2021 investigation were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The borehole locations and ground surface elevations completed during the July 2010 investigation were provided by Annis, O'Sullivan, Vollebekk Ltd. The locations of the boreholes, and ground surface elevation at each borehole location, are presented on Drawing PG7871-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

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

Two (2) soil samples was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The northern portion of the subject site is occupied by a multi-storey building with associated asphalt paved access lanes, parking areas and landscaped areas. The southern portion of the site is vacant and undeveloped with sparse areas of mature trees which generally follow the southern, eastern and western property boundaries.

The site is bordered to the east by a multi-use pedestrian pathway followed by single family residential dwellings, to the south by Byron Avenue, to the north by a multi-storey mixed-use building and to the west by an elementary school and single-family residential dwellings. The ground surface across the property slopes gently downward from south to north at approximate geodetic elevations of 70 m to 68 m.

Based on available historical aerial photographs, the southern portion of the subject site was occupied by a park as recently as 2008. Between approximately 2009 and 2011, the western portion of the park was used as a staging and stockpile area for fill and construction material and expanded to cover the entire site by 2013 to 2014. Historical aerial photographs of the subject site and its surroundings are provided on Figures 2, 3 and 4.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the subject site consists of topsoil and/or fill underlain by a thin layer of silty clay and glacial till. The fill material was generally observed to consist of brown silty sand with gravel, clay, crushed stone as well as topsoil and organics, and extended to maximum depths of 0.7 to 2.3 m below the existing ground surface.

A thin layer of stiff brown silty clay to clayey silt with some sand and trace organics was encountered below the fill and/or topsoil layers at boreholes BH 1-21 and BH 3-21, extending to maximum depths of 1.9 and 3.0 m, respectively. Additionally, a firm to stiff grey silty clay was encountered underlying the fill material at all test pit locations and generally extended to the maximum depth of the test pits. At test pit TP 2-23, the silty clay was noted to transition to soft to firm, with trace amounts of sand and gravel.

A compact, brown silty sand with gravel was encountered below the fill later at test pit TP 1-23, and extended to an approximate depth of 2.3 m below the existing ground surface. A loose to compact, grey silty sand was encountered below the silty clay layer at test pit TP 5-23.

A glacial till deposit was encountered underlying the above-noted soils at approximate depths ranging from 0.1 to 3.0 within the boreholes and was generally observed to consist of loose to very dense brown and/or grey silty sand with clay, gravel cobbles and boulders. At boreholes BH 1, BH 2, BH 4 and BH 5, the glacial till deposit was observed to transition from brown to grey in colour at approximate depths ranging from 2.9 to 3.7 m below ground surface. Running sand was encountered at boreholes BH 1-21 and BH 2-21 at approximate 4.5 to 7.6 m below the existing ground surface.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each borehole location.

Bedrock

The bedrock was cored at boreholes BH 1-21, BH 2-21, BH 3-21, BH 2, BH 3, and BH 4, and based on the RQDs of the recovered bedrock core, was generally noted to consist of limestone of poor to fair in quality in the upper 0.5 to 0.8 m, becoming good to excellent with depth. The bedrock was cored to a maximum depth of 15.0 m below the existing ground surface.

Practical refusal to augering was encountered at BH 1 and BH 5 at a depth of 8.7 and 9.8 m, respectively on inferred bedrock.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and dolomite of the Ottawa Formation.

4.3 Groundwater

Groundwater levels were measured in the installed monitoring wells and piezometers. The observed groundwater levels are summarized in Table 1 below.

Table 1 - Summary of Groundwater Levels				
Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Levels		Date Recorded
		Depth (m)	Elevation (m)	
BH 1-21*	70.08	3.86	66.22	February 8, 2021
BH 2-21*	68.53	3.05	65.48	February 8, 2021
BH 3-21*	68.31	6.54	61.77	February 8, 2021
BH 1	68.42	2.48	65.94	July 20, 2010
BH 2	68.31	2.90	65.41	July 20, 2010
BH 3	67.85	2.22	65.63	July 20, 2010
BH 4	69.04	1.69	67.35	July 20, 2010
BH 5	68.81	1.02	67.79	July 20, 2010
Note: '*' – Denotes Monitoring Well				

It should be noted that water levels recorded in the standpipe or groundwater monitoring wells can result in higher than normal groundwater level readings as a result of water that may become trapped within backfilled boreholes. The groundwater level can also be determined based on field observations, such as moisture levels and colouring. Based on these observations, the long-term groundwater level is anticipated to be encountered between an approximate 3 to 4 m depth below existing ground surface.

However, it should be noted that the groundwater is subject to seasonal fluctuations and therefore, the groundwater level could vary at the time of construction.

Hydraulic Conductivity Analysis

Following the completion of the slug testing, the test data was analyzed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous and isotropic aquifer of infinite extent with zero-storage assumption, and a screen length significantly greater than the monitoring well diameter. The assumption regarding aquifer storage is considered to be appropriate for groundwater flow through the overburden aquifer. The assumption regarding screen length and well diameter is considered to be met based on a screen length of 3 m and a diameter of 0.051 m.

While the idealized assumptions regarding aquifer extent, homogeneity, and isotropy are not strictly met in this case (or in any real-world situation), it has been our experience that the Hvorslev method produces effective point estimates of hydraulic conductivity in conditions similar to those encountered at the subject site.

Hvorslev analysis is based on the line of best fit through the field data (hydraulic head recovery vs. time), plotted on a semi-logarithmic scale. In cases where the initial hydraulic head displacement is known with relative certainty, such as in this case where a physical slug has been introduced, the line of best fit is considered to pass through the origin.

Based on the above test methods, the monitoring wells screened in the glacial till displayed hydraulic conductivity values ranging from **1.33×10^{-5} to 3.02×10^{-5} m/sec**. The values measured within the monitoring wells are generally consistent with similar material Paterson has encountered on other sites and typical published values for glacial till with a silty sand matrix. These values typically range from 1×10^{-5} to 1×10^{-6} m/sec for a glacial till deposit with a silty sand matrix. The range in hydraulic conductivity values is due to the variability of the material in the deposit. The results of the hydraulic conductivity testing are presented in Appendix 1.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed residential development. It is recommended that the proposed buildings be founded on conventional spread footings bearing on the undisturbed compact brown silty sand, firm to stiff brown to grey silty clay or on compact to very dense glacial till.

It is expected that the proposed multi-storey building will founded at a lower elevation than the existing heritage building and subsequently within the lateral support zone of the existing footings. As a result, the existing footings may require an underpinning program or temporary shoring system consisting of a secant pile wall, where the proposed structure is located directly against the existing heritage building.

Further, where temporary shoring is needed beyond the existing building, but where the excavation still extends within the lateral support zone of the existing building footings, the temporary shoring system should consist of concrete secant pile wall.

Due to the presence of a silty clay deposit within a portion of the site, areas within the proposed development will be subjected to grade raise restrictions. Our permissible grade raise recommendations are discussed in Section 5.3 and presented on Drawing PG7871-2 – Permissible Grade Raise Plan.

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

5.2 Grading and Preparation

Stripping Depth

Due to previous site work and land use, a significant thickness of topsoil and deleterious fill was observed across the majority of the subject site. Topsoil and fill, such as those containing significant organic or deleterious material, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants, such as foundation walls should be excavated to a minimum of 1 m below final grade.

Fill Placement

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

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level of areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of the 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 composite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on an undisturbed, compact to very dense glacial till can be designed using a bearing resistance value at serviceability limit states (SLS) of **350 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **500 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

Footings placed on an undisturbed, compact silty sand, compact sandy silt and/or firm to stiff silty clay can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

If the silty sand or glacial till bearing medium is encountered in a loose state of compactness at the time of construction, the bearing surface should be proof-rolled using a suitable compaction material making several passes. The proof-rolling should be completed under dry conditions and above freezing temperatures and approved by the Paterson 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 prior to the placement of concrete for footings.

Footings placed on an undisturbed soil bearing surface and designed using the bearing resistance values at SLS 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. Above the groundwater level, adequate lateral support is provided to the in-situ bearing medium soils 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.

Permissible Grade Raise Recommendations

Due to the presence of the silty clay deposit encountered within areas of the subject site, a permissible grade raise restriction of **1.5 m** is recommended for all structures placed on a silty clay bearing medium. The recommended grade raise restrictions are shown on PG7871-2 – Permissible Grade Raise Plan included in Appendix 2.

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.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to determine the applicable seismic site designation for the proposed development in accordance with the Ontario Building Code 2024. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave interpretation are presented in Appendix 2.

Field Program

The shear wave testing was located towards the southern half of the site, as presented in Drawing PG7871-1 - Test Hole Location Plan presented in Appendix 2. Paterson field personnel placed 24 horizontal geophones in a straight line in roughly a north-south orientation towards the southern side of the site. The

4.5 Hz. horizontal geophones were mounted to the surface by means of a 75 mm ground spike attached to the geophone land case. The geophones were spaced at 3 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between 4 to 8 times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at the centre of the geophone array and at 3, 4.5 and 15 m away from the first and last geophone.

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, reflected, 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 soil profile, 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.

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

The glacial till and bedrock velocities were interpreted to be **281 m/s** and **1,973 m/s**, respectively.

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2024. If the building is founded on glacial till, approximately 3 m above the bedrock surface, the following equation applies:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{3\ m}{281\ m/s} + \frac{27\ m}{1973\ m/s}\right)}$$

$$V_{s30=}\ 1231\ m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs30, for foundations placed on the glacial till deposit, and no more than 3 m from the bedrock surface, is **1,231 m/s**. Therefore, a **Site Class $\mathbf{X_{1231}}$** is applicable for design of the proposed buildings, as per Table 4.1.8.4.A of the OBC 2024. Based on Paterson's review of the in-situ soil characteristics, the soils underlying the subject site are not considered susceptible to liquefaction.

5.5 Floor Slab Construction

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the native soil and/or approved fill is considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.
For structures with slab-on-grade construction, it is recommended that the upper 200 mm of sub-slab fill consists of OPSS Granular A crushed stone. For structures with basement slabs, it is recommended that the upper 300 mm of sub-floor fill consists of 19 mm clear crush stone.

All backfill material within the footprint of the proposed buildings should be placed in a maximum 300 mm thick loose layers and compacted to a minimum of 98% of the material's SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

It is anticipated that the underground level for the proposed multi-storey apartment building will primarily consist of parking areas and the rigid pavement structure recommendation provided in Section 5.8 will be applicable. However, if storage or other uses of the lower level are considered where a concrete floor slab will be constructed, the upper 300 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

In consideration of the groundwater conditions at the site and the depth of excavation, a sub-slab drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided under the lowest level floor slab. This is discussed further in Section 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed building. However, in our opinion, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3.

The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

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

Static Earth Pressures

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

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

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. 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 $0.375 \cdot a \cdot H^2/g$ where:

$a_c = (1.45 - a_{max}/g)amax$
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 subject site is 0.30g according to OBC 2024. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o \cdot \gamma \cdot H^2$, 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 forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2024.

5.8 Pavement Design

Underground Parking Level

For design purposes, it is recommended that the rigid pavement structure for the underground parking level of the proposed building consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The following rigid pavement structure is recommended.

Table 2 – Recommended Rigid Pavement Structure – Lowest Underground Parking Level	
Thickness (mm)	**Material Description**
150	**Exposure Class C2 – 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** – OPSS Granular A Crushed Stone
SUBGRADE – Existing imported fill, or OPSS Granular B Type I or II material placed over in bedrock.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level.

The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Podium Deck Area

It is anticipated that the podium deck structure will be provided car only parking areas, access lanes, fire truck lanes and loading areas. Based on the concrete slab subgrade, the pavement structure indicated in the Tables 3 and 4 may be considered for design purposes:

Table 3 – Recommended Pavement Structure – Car-Only Parking Areas (Podium Deck)

Thickness (mm)	Material Description
50	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
200**	**Base** – OPSS Granular A Crushed Stone
See Below*	**Thermal Break*** – Rigid insulation (See Paragraph Below)
n/a	**Waterproofing Membrane and Protection Board**
SUBGRADE – Reinforced Concrete Podium Deck *If specified by others, not required from a geotechnical perspective **Thickness is dependent on grade of insulation as noted in proceeding paragraph	

Table 4 – Recommended Pavement Structure – Access Lanes, Fire Truck Lanes, Ramp and Heavy Truck Parking Areas (Podium Deck)

Thickness (mm)	Material Description
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course** – HL-8 or Superpave 19.0 Asphaltic Concrete
300**	**Base** – OPSS Granular A Crushed Stone
See Below*	**Thermal Break*** – Rigid insulation (See Paragraph Below)
n/a	**Waterproofing Membrane and IKO Protection Board**
SUBGRADE – Reinforced Concrete Podium Deck *If specified by others, not required from a geotechnical perspective **Thickness is dependent on grade of insulation as noted in proceeding paragraph	

The transition between the pavement structure over the podium deck subgrade and soil subgrade beyond the footprint of the podium deck is recommended to be transitioned to match the pavement structures provided in the following section. For this transition, a 5H:1V is recommended between the two subgrade surfaces. Further, the base layer thickness should be increased to a minimum thickness of 500 mm below the top of the podium slab a minimum of 1.5 m from the face of the foundation wall prior to providing the recommended taper.

Should the proposed podium deck be specified to be provided a thermal break by the use of a layer of rigid insulation below the pavement structure, its placement within the pavement structure is recommended to be as per the above-noted tables. The layer of rigid insulation is recommended to consist of a DOW Chemical High-Load 100 (HI-100), High-Load 60 (HI-60) or High Load (HI-40). The pavement structures base layer thickness will be dependent on the grade of insulation considered for this project and should be reassessed by the geotechnical consultant once pertinent design details have been prepared.

The higher grades of insulation have more resistance to deformation under wheel-loading and require less granular cover to avoid being crushing by vehicular loading. It should be noted that SM (Styrofoam) rigid insulation is not considered suitable for this application.

Pavement Structure Over Overburden

Beyond the podium deck, the following pavement structures may be considered for car only parking and heavy traffic areas. The proposed pavement structures are shown in Tables 5 and 6 below:

Table 5 – 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, bedrock or fill.	

Table 6 – Recommended Pavement Structure – Access Lanes and Heavy Loading Area	
Thickness (mm)	**Material Description**
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – HL-8 or Superpave 19 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, bedrock 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 I or II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment, noting that excessive compaction can result in subgrade softening.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for each proposed structure which has below-grade space. The system should consist of a 150 mm diameter perforated and corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, which is placed at the footing level around the exterior perimeter of the structure. The pipe should have positive outlet, such as a gravity connection to the storm sewer, or to the sump pit where sump pumps are proposed at the residential dwellings.

Based on the anticipated depth of the proposed buildings, precipitation from a five-year one-hour duration storm event and the hydrogeological properties described in Section 4.3, groundwater and surface water contributions to the perimeter foundation drainage system are estimated to be approximately 60,000 L/day. Larger contributions could be encountered during precipitation events larger than the design storm event or during spring freshet. It is recommended that a factor of safety be applied to the above noted volume to account for unforeseen circumstances. It is further recommended to direct surface water runoff away from the foundations as well as avoid snow storage in close proximity to the proposed buildings.

Underslab Drainage System

An underslab drainage system is recommended to control water infiltration below the underground parking level slab. For preliminary design purposes, it is recommended that 150 mm perforated pipes be placed at approximate 6 m centres underlying the underground parking level slab. The spacing of the underslab drainage system should be confirmed by the geotechnical consultant at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

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 Delta Drain 6000, connected to the perimeter foundation drainage system.

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent thickness of soil cover and foundation insulation, should be provided in this regard.

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

6.3 Excavation Side Slopes and Temporary Shoring

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

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

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

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

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

Temporary Shoring

For preliminary design purposes, the temporary system may consist of secant wall and/or soldier pile and lagging system.

To lessen impacts to the existing heritage building, it is recommended that the temporary shoring system located within the lateral support zone of the existing building should consist of a concrete secant pile wall due. Elsewhere, outside of the lateral support zone of the existing building, shoring may consist of a conventional soldier pile and lagging system.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's structural designer prior to implementation.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced. Earth pressures acting on the shoring system may be calculated using the parameters provided in Table 5.

Table 5 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (Ka)	0.33
Passive Earth Pressure Coefficient (Kp)	3
At-Rest Earth Pressure Coefficient (Ko)	0.5
Unit Weight , kN/m3	20
Submerged Unit Weight , kN/m3	13

Underpinning Program

As an alternative to the use of a secant pile wall in areas where the proposed building is located in proximity to the existing heritage building, an underpinning program could be implemented to safely transfer the existing building loads down to a lower founding elevation. Once details for the proposed building are finalized, it is recommended that details of the underpinning program be prepared in conjunction with the project's structural engineer, if required. It is anticipated that underpinning could consist of conventional concrete piers installed in piano-key fashion.

6.4 Pipe Bedding and Backfill

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

The pipe bedding for sewer and water pipes should consist of at least 150 mm of OPSS Granular A material. Where the bedding is located on the grey silty clay, the thickness of the bedding material should be increased to 300 mm. The bedding should extend to the spring line of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's standard Proctor maximum dry density.

Cover material, from the spring line to a minimum of 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 and compacted to a minimum of 99% of its SPMDD.

It should generally be possible to re-use the site generated fill materials (moist, not wet) above the cover material if excavation and filling operations are carried out in dry and non-freezing weather conditions. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

Where hard surface areas are considered above the trench backfill, the backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 98% of the material's SPMDD. All cobbles larger than 200 mm in their longest direction should be segregated from re-use as trench backfill.

6.5 Groundwater Control

Based on our observations, it is anticipated that groundwater infiltration into the excavation should be low to moderate and controllable using open sumps. 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.

Permit to Take Water

Under the current regulations enacted by the Ministry of Environment, Conservation and Parks (MECP), any dewatering in excess of 50,000 L/day requires a registration on the Environmental Activity and Sector Registry (EASR), so long as that dewatering is related to construction. If the dewatering is not related to construction, a Permit to Take Water obtained from the MECP will be required.

In the event that an EASR is required to facilitate dewatering of the proposed development, a minimum of three 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. Should a Permit to Take Water be required, a minimum of five to six months should be allotted for completion of the permit, due to the minimum review period imposed by the MECP.

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 using 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.

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 the following material testing and observation program be performed by the geotechnical consultant.

- ❑ Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.
- ❑ Periodic observation of the underpinning program for the adjacent heritage building during the excavation program.
- ❑ Review detailed grading plan(s) from a geotechnical perspective.
- ❑ 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 the completion of a satisfactory inspection program by the geotechnical consultant.

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

8.0 Statement of Limitations

The recommendations provided 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 Concorde Properties, 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.

LICENSED PROFESSIONAL ENGINEER
February 24, 2026
K. A. PICKARD
100531344
PROVINCE OF ONTARIO

Mrunmayi Anvekar, M.Eng.

Kevin A. Pickard, P.Eng.

Report Distribution:

- ❑ Concorde Properties (1 email copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

HYDRAULIC CONDUCTIVITY RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2021 February 1

FILE NO. PG2159

HOLE NO. BH 1-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with gravel, crushed stone, some cobbles, trace organics	0.71	AU	1		
Stiff brown **SILTY CLAY** with some sand, trace organics	1.93	SS	2	67	14
		SS	3	88	72
GLACIAL TILL: Grey silty sand with gravel, some clay, trace cobbles occasional boulders	3.66	SS	4	79	+50
		SS	5	0	+50
GLACIAL TILL: Dense brown silty sand with gravel, some cobbles - Some course sand by 4.5 m depth - Some running sand encountered between 4.5 m and 7.6 m	7.62	SS	6	63	+50
		SS	7	54	+50
		SS	8	0	+50
		SS	9	75	+50
GLACIAL TILL: Compact grey silty sand, some gravel, cobbles and occasional boulders	12.27	SS	10	54	16
		RC	1	100	60
BEDROCK: Good to excellent quality grey Limestone	14.99	RC	2	100	100
End of Borehole					

(GWL @ 3.86 m depth - Feb 8, 2021)

DEPTH (m)	ELEV. (m)
0	70.08
1	69.08
2	68.08
3	67.08
4	66.08
5	65.08
6	64.08
7	63.08
8	62.08
9	61.08
10	60.08
11	59.08
12	58.08
13	57.08
14	56.08

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

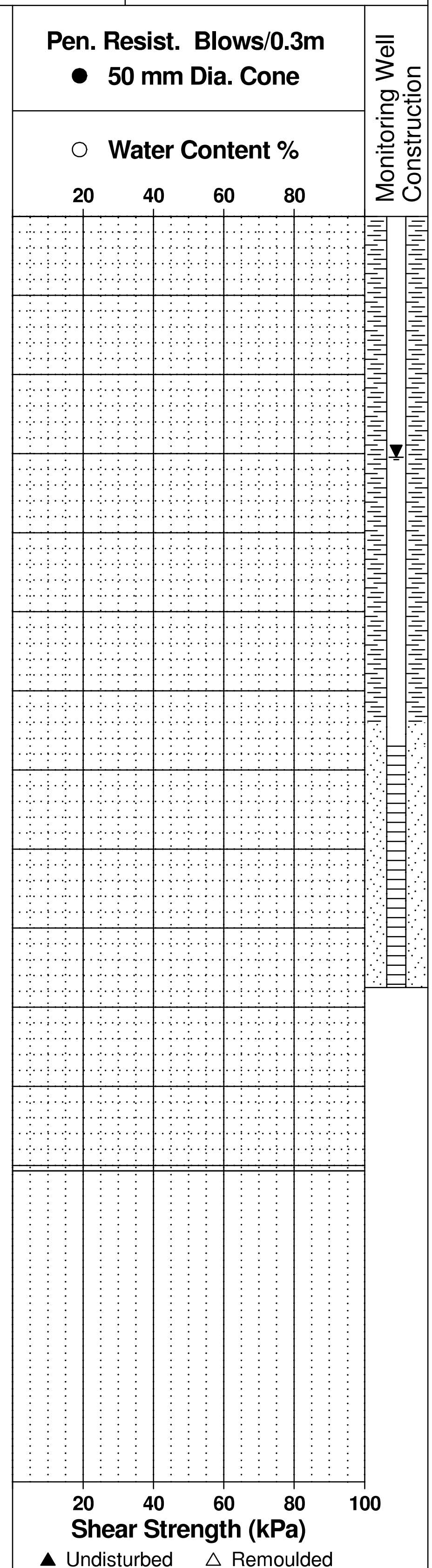
BORINGS BY CME-55 Low Clearance Drill

DATE 2021 February 1

FILE NO. PG2159

HOLE NO. BH 2-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.53
FILL: Brown silty sand with crushed stone trace organics and topsoil		AU	1				
		SS	2	83	7	1	67.53
1.35							
TOPSOIL some gravel							
1.52							
GLACIAL TILL: Compact to dense, grey silty sand with clay, gravel, trace cobbles - Decreasing clay content with depth		SS	3	58	13	2	66.53
		SS	4	0	+50		
		SS	5	67	65	3	65.53
		SS	6	63	51	4	64.53
4.57							
GLACIAL TILL: Grey silty sand trace gravel - Some running sand encountered from 4.5 m to 6.1 m depth		SS	7	4	29	5	63.53
		SS	8	92	28	6	62.53
- Some gravel, cobbles and occasional boulders by 6.1 m depth		SS	9	21	36		
						7	61.53
						8	60.53
						9	59.53
9.83							
BEDROCK: Poor to good quality grey Limestone		RC	1	100	39	10	58.53
		RC	2	100	69	11	57.53
12.07						12	56.53
End of Borehole (GWL @ 3.05 m depth - Feb 8, 2021)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2021 February 1

FILE NO. PG2159

HOLE NO. BH 3-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with crushed stone	0.30	AU	1		
FILL: Brown silty sand trace clay and gravel	0.66	SS	2	42	10
FILL: Brown silty sand some clay	1.52	SS	3	46	3
TOPSOIL	1.73				
Brown **CLAYEY SILT** with organics	2.13	SS	4	100	2
Stiff brown **SILTY CLAY**	2.97				
GLACIAL TILL brown silty sand, trace gravel		SS	5	63	37
		SS	6	100	56
		SS	7	79	63
- Occasional cobbles by 5.4 m depth		SS	8	17	+50
	7.62	SS	9	38	+50
GLACIAL TILL: Brown silty sand to sandy silt with gravel, cobbles and occasional boulders	9.75	SS	10	33	+50
BEDROCK: Fair to good quality grey Limestone		RC	1	100	53
	12.67	RC	2	97	76
End of Borehole					
(GWL @ 6.54 m depth - Feb 9, 2021)					

DEPTH (m)	ELEV. (m)
0	68.31
1	67.31
2	66.31
3	65.31
4	64.31
5	63.31
6	62.31
7	61.31
8	60.31
9	59.31
10	58.31
11	57.31
12	56.31

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG2159

REMARKS

HOLE NO. BH 1

BORINGS BY CME 55 Power Auger **DATE** 13 July 2010

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.42
TOPSOIL 0.13		AU	1				
GLACIAL TILL: Dense to very dense, brown gravelly sand with silt, cobbles and boulders		SS	2		34	1	67.42
		SS	3	58	82	2	66.42
		SS	4	56	50+		
2.90						3	65.42
GLACIAL TILL: Loose to compact, grey silty sandy clay with gravel, cobbles and boulders		SS	5	100	1		
		SS	6	42	2	4	64.42
		SS	7	83	2	5	63.42
		SS	8	100	1	6	62.42
		SS	9	33	16	7	61.42
		SS	10	75	16	8	60.42
8.74							

End of Borehole

Practical refusal to augering @ 8.74m depth

(GWL @ 2.48m-July 20/10)

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG2159

REMARKS

HOLE NO. BH 2

BORINGS BY CME 55 Power Auger **DATE** 13 July 2010

SOIL DESCRIPTION	Depth to base (m)	TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.31
TOPSOIL	0.10	AU	1				
GLACIAL TILL: Very dense, brown gravelly sand with silt, cobbles and boulders	3.66	SS	2	50	91	1	67.31
		SS	3	91	50+	2	66.31
		SS	4	100	50+		
		SS	5	80	50+	3	65.31
GLACIAL TILL: Loose, grey silty sandy clay with gravel and cobbles	7.00	SS	6	50	6	4	64.31
		SS	7	67	4	5	63.31
		SS	8	62	2	6	62.31
		SS	9	38	9	7	61.31
GLACIAL TILL: Dense, grey silty sand with gravel and cobbles	9.02	SS	10	50	32	8	60.31
BEDROCK: Grey limestone	10.11	RC	1	100	80	9	59.31
End of Borehole						10	58.31

(GWL @ 2.9m depth based on field observations)

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

Piezometer Construction

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

patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG2159

REMARKS

HOLE NO. BH 3

BORINGS BY CME 55 Power Auger **DATE** 13 July 2010

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.30	AU	1		
GLACIAL TILL: Loose, grey silty clay with sand and gravel	1.45	SS	2	29	2
GLACIAL TILL: Compact to very dense, grey silty sand with gravel, cobbles and boulders	8.69	SS	3	100	
		SS	4	100	26
		SS	5	50	
		SS	6	100	50+
		SS	7	100	12
		SS	8	100	50+
		SS	9	100	50+
		SS	10	91	50+
		SS	11	79	53
BEDROCK: Grey limestone	10.21	RC	1	100	29
End of Borehole					
(GWL @ 2.22m-July 20/10)					

DEPTH (m)	ELEV. (m)
0	67.85
1	66.85
2	65.85
3	64.85
4	63.85
5	62.85
6	61.85
7	60.85
8	59.85
9	58.85
10	57.85

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG2159

REMARKS

HOLE NO. BH 4

BORINGS BY CME 55 Power Auger **DATE** 13 July 2010

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	69.04
TOPSOIL 0.30	AU	1				
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders, trace clay	SS	2		43	1	68.04
	SS	3	75	52	2	67.04
	SS	4	100	64	3	66.04
- grey by 3.7m depth	SS	5	100	50+	4	65.04
	SS	6		50+	5	64.04
	SS	7	100	50+	6	63.04
	SS	8	100	58	7	62.04
	SS	9		73	8	61.04
					9	60.04
10.69					10	59.04
BEDROCK: Grey limestone	RC	1	100	21	11	58.04
12.22					12	57.04
End of Borehole						
(GWL @ 1.69m-July 20/10)						

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup
Consulting Engineers

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 13 July 2010

FILE NO. PG2159

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.81
TOPSOIL 0.30		AU	1				
		SS	2	67	17	1	67.81
		SS	3	78	50+		
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders, trace clay						2	66.81
- grey by 3.0m depth						3	65.81
		SS	4		37		
		SS	5	75	74	4	64.81
		SS	6	100	47	5	63.81
						6	62.81
		SS	7	100	50+		
						7	61.81
		SS	8		51	8	60.81
						9	59.81
9.83		SS	9	100	50+		

End of Borehole

Practical refusal to augering @ 9.83m depth

(GWL @ 1.02m-July 20/10)

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

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Phase II Enviromental Site Asssesment
114 Richmond Road
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

FILE NO. PE1699

REMARKS

HOLE NO. TP 1-23

BORINGS BY Excavator **DATE** May 18, 2023

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.55
FILL: Gravel with organics, brown silty sand, trace to some topsoil 0.20			1				
FILL: Compact brown silty sand with gravel, some cobbles, trace clay. topsoil, organics and asphalt 0.75			2				
Compact, brown **SILTY SAND** with gravel, with some gravel 2.30			3			1	68.55
			4			2	67.55
Light brown to grey **SILTY CLAY** 2.60							
Firm to stiff. light grey **SILTY CLAY** 4.00			5			3	66.55
			6				
			7			4	65.55
End of Test Pit							

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**

20 40 60 80

100 200 300 400 500

RKI Eagle Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Phase II Enviromental Site Asssesment
114 Richmond Road
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

FILE NO. PE1699

REMARKS

HOLE NO. TP 2-23

BORINGS BY Excavator

DATE May 18, 2023

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.15
FILL: Compact to dense, brown silty sand with gravel, some cobbles, trace organics (0.60)			1				
FILL: Compact to dense silty sand with gravel, some cobbles, trace organics (1.40)			2			1	68.15
FILL: Compact silty sand with gravel, some cobbles, trace clay and topsoil, trace organics and asphalt (2.00)			3			2	67.15
FILL: Compact silty sand with gravel, some cobbles, trace clay and topsoil, trace organics and asphalt (2.35)			4				
Firm to stiff. grey **SILTY CLAY** (3.70)			5			3	66.15
Soft to firm. grey **SILTY CLAY** trace sily, sand and gravel (4.00)			6			4	65.15
End of Test Pit							

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**
20 40 60 80

100 200 300 400 500
RKI Eagle Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Phase II Enviromental Site Asssesment
114 Richmond Road
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

REMARKS

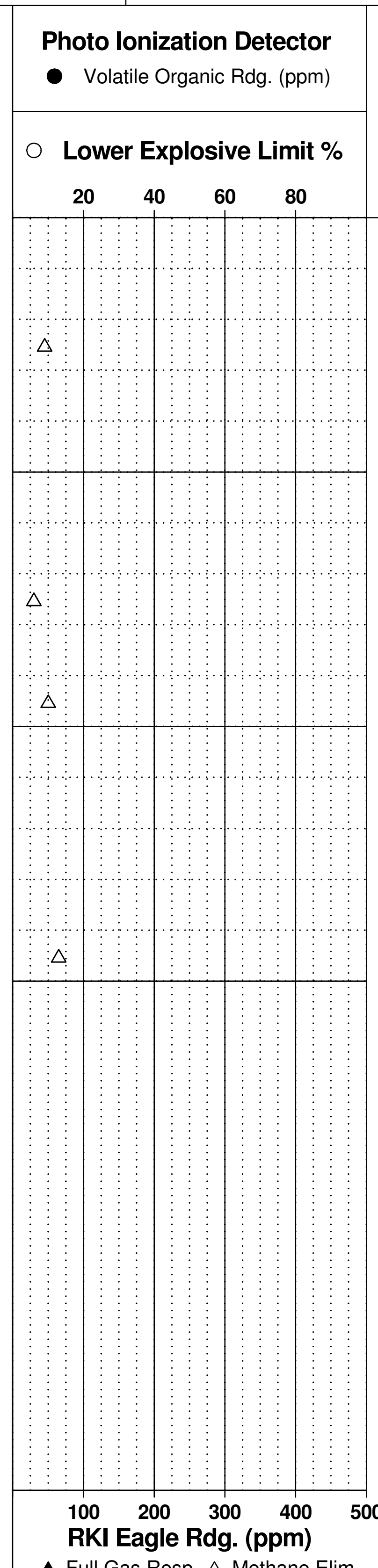
BORINGS BY Excavator

DATE May 18, 2023

FILE NO. PE1699

HOLE NO. TP 3-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.99
FILL: Compact to dense, brown silty sand with gravel, some cobbles, trace organics			1			1	67.99
1.30							
FILL: Dense, brown silty sand with gravel, some cobbles, trace organics, brock, wood, concrete, topsoil and clay			2				
1.70							
FILL: Compact silty sand with some clay, topsoil, trace gravel, wood and concrete			3			2	66.99
2.30							
Firm. grey **SILTY CLAY**			4				
3.00						3	65.99
End of Test Pit							



Monitoring Well Construction

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Phase II Enviromental Site Asssesment
114 Richmond Road
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

REMARKS

BORINGS BY Excavator

DATE May 18, 2023

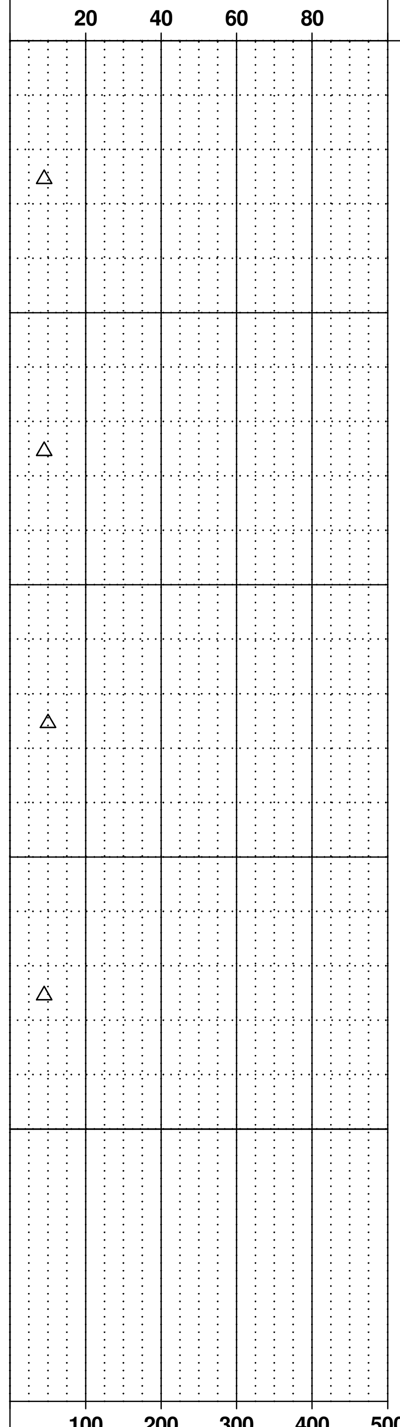
FILE NO. PE1699

HOLE NO. TP 4-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.96
FILL: Dense to very dense gravel, with light brown silty sand with some cobbles and occasional organics			1				
1.00						1	67.96
FILL: Compact, brown, silty sand with some gravel, topsoil, trace clay, concrete organics and brick			2				
						2	66.96
2.30							
Firm to stiff. light grey **SILTY CLAY**			3				
						3	65.96
			4				
4.00						4	64.96
End of Test Pit							

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**



RKI Eagle Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Phase II Enviromental Site Asssesment
114 Richmond Road
Ottawa, Ontario

DATUM Elevations are referenced to a geodetic datum

REMARKS

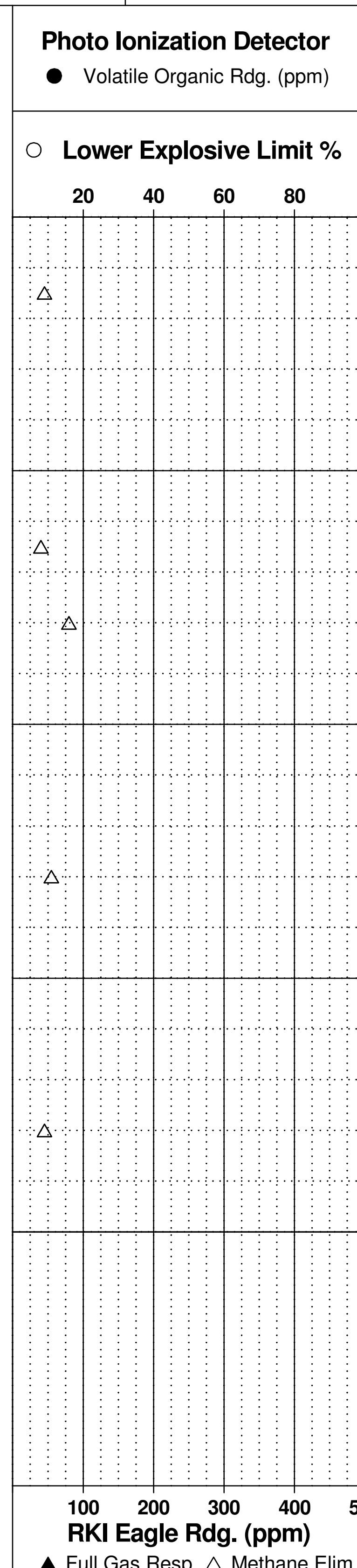
BORINGS BY Excavator

DATE May 18, 2023

FILE NO. PE1699

HOLE NO. TP 5-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	68.24
FILL: Compact brown silty sand, with gravel some topsoil, trace cobbles			1				
1.00						1	67.24
			2				
1.40							
FILL: Compact topsoil with brown silty sand, some gravel, trace organics and cobbles			3				
						2	66.24
Firm to stiff. light grey **SILTY CLAY**			4				
3.00						3	65.24
Loose to compact, light grey **SILTY SAND**			5				
4.00						4	64.24
End of Test Pit							



Monitoring Well Construction

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

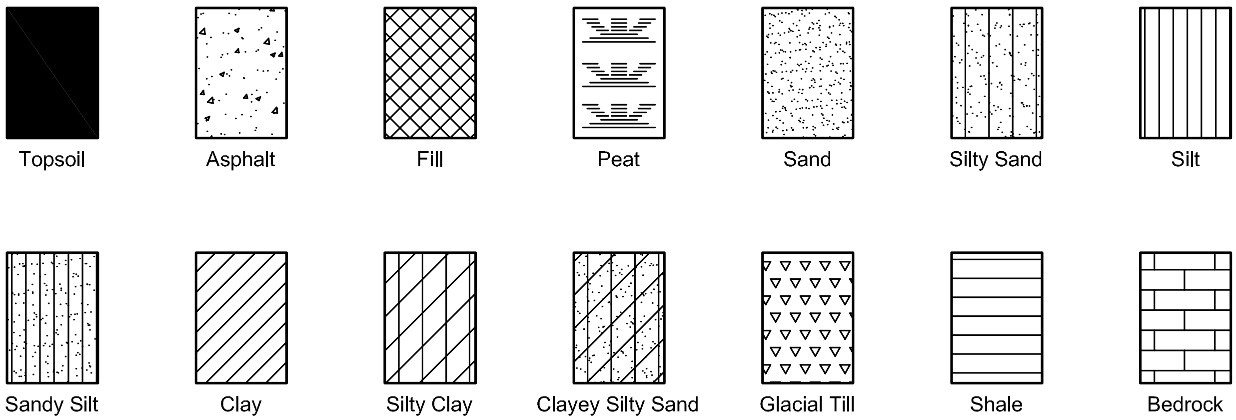
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

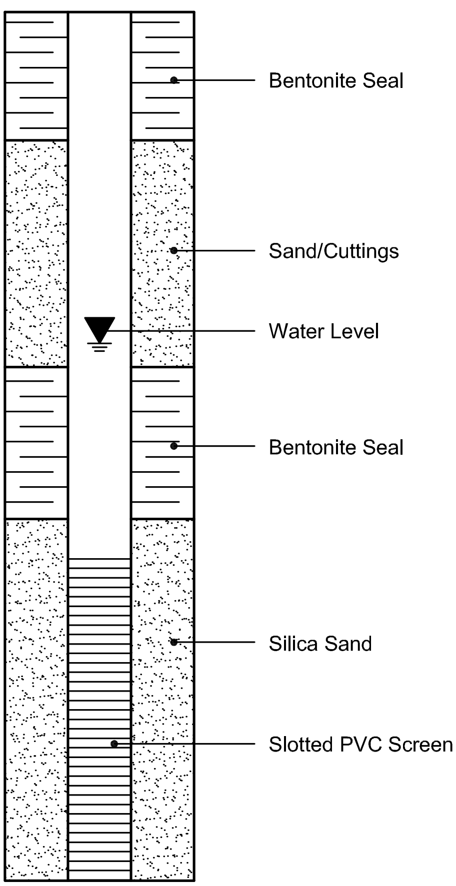
SYMBOLS AND TERMS (continued)

STRATA PLOT

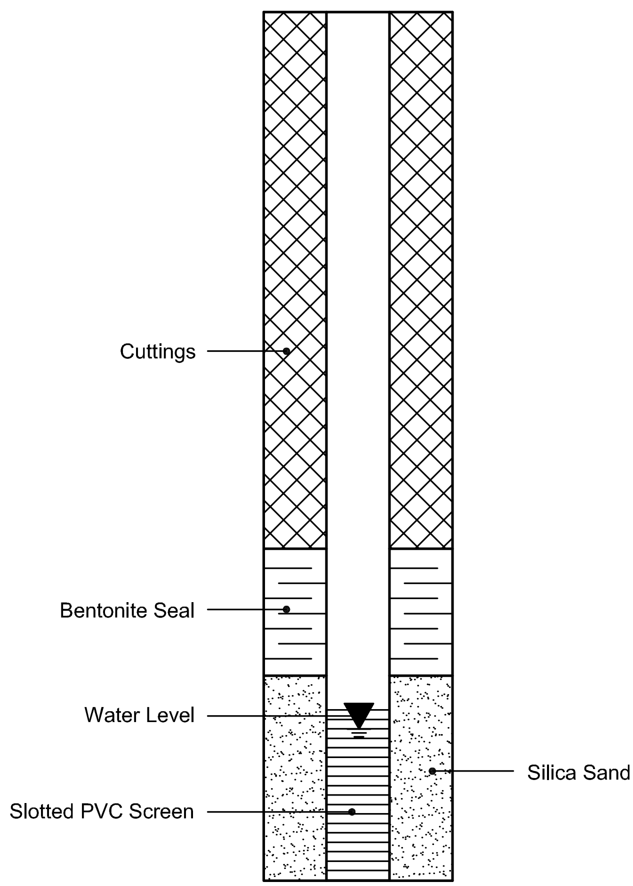


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PARACEL LABORATORIES LTD.

Order #: 1030028

Certificate of Analysis

Report Date: 21-Jul-2010
Order Date:19-Jul-2010

Client: **Paterson Group Consulting Engineers**
Client PO: 8997
Project Description: PG2159

	Client ID:	BH3 SS8	-	-	-
	Sample Date:	14-Jul-10	-	-	-
	Sample ID:	1030028-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	82.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.93	-	-	-
Resistivity	0.10 Ohm.m	59.8	-	-	-
Anions					
Chloride	5 ug/g dry	32	-	-	-
Sulphate	5 ug/g dry	36	-	-	-

Certificate of Analysis

Report Date: 08-Feb-2021

Client: **Paterson Group Consulting Engineers**

Order Date: 2-Feb-2021

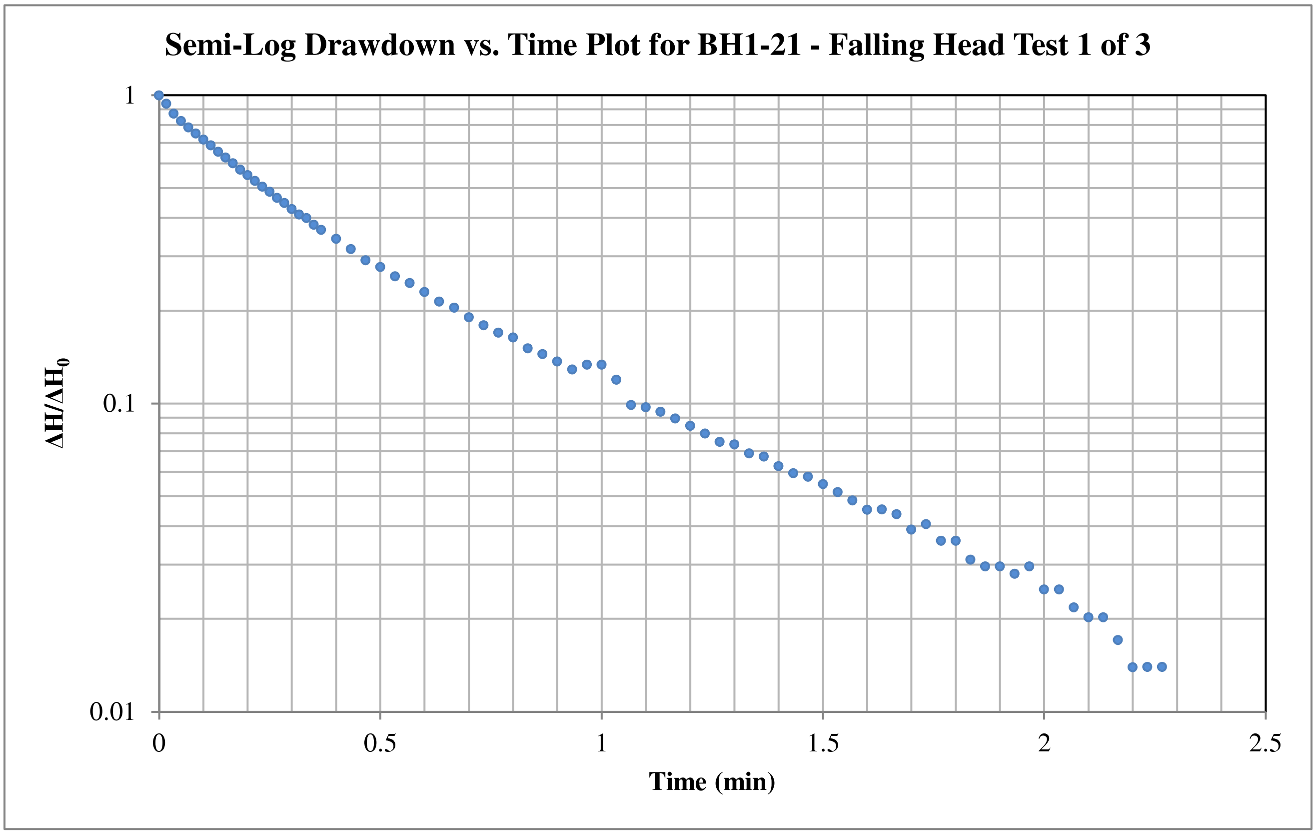
Client PO: 31891

Project Description: PG2159

	Client ID:	BH1-21-SS4	-	-	-
	Sample Date:	29-Jan-21 13:00	-	-	-
	Sample ID:	2106205-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.39	-	-	-
Resistivity	0.10 Ohm.m	65.1	-	-	-
Anions					
Chloride	5 ug/g dry	15	-	-	-
Sulphate	5 ug/g dry	85	-	-	-

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH1-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

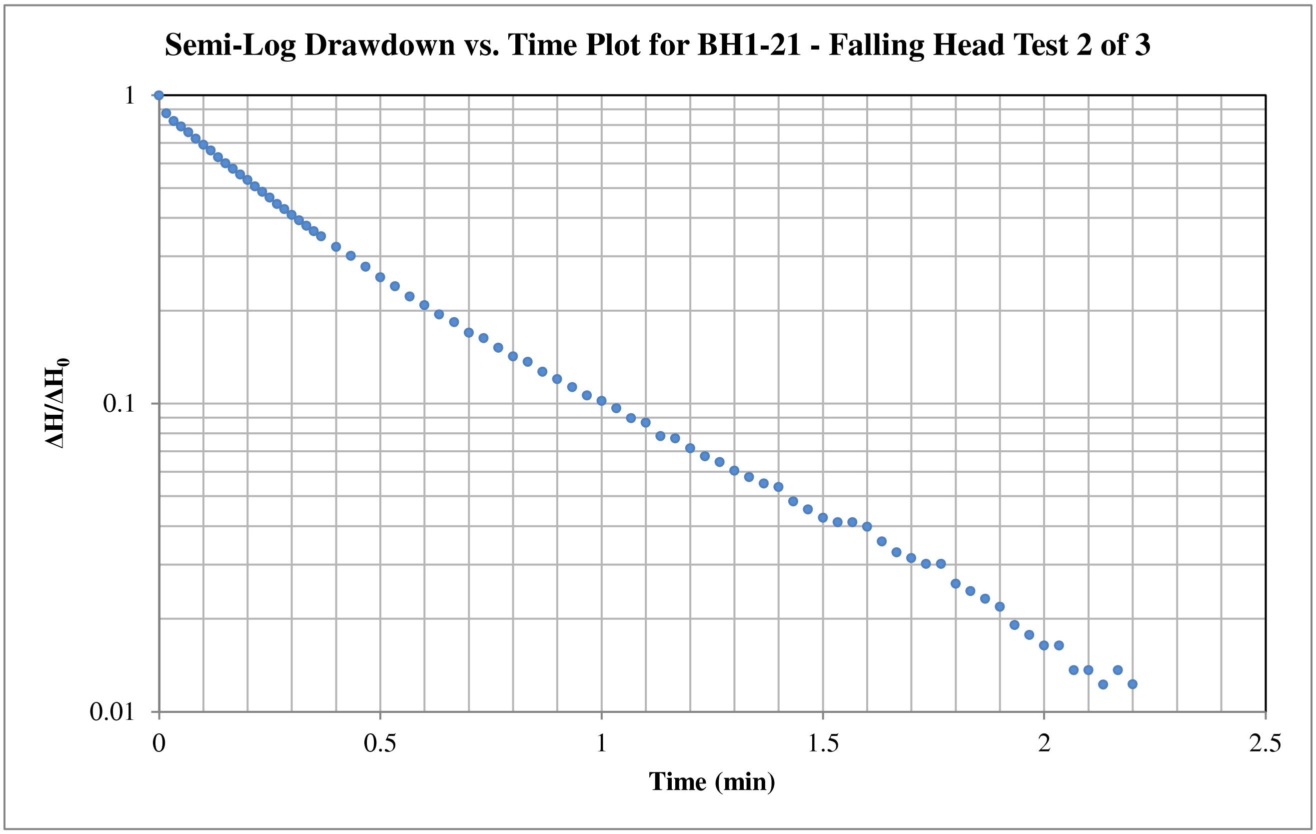
Data Points (from plot):

t^*: 0.363 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.36E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH1-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

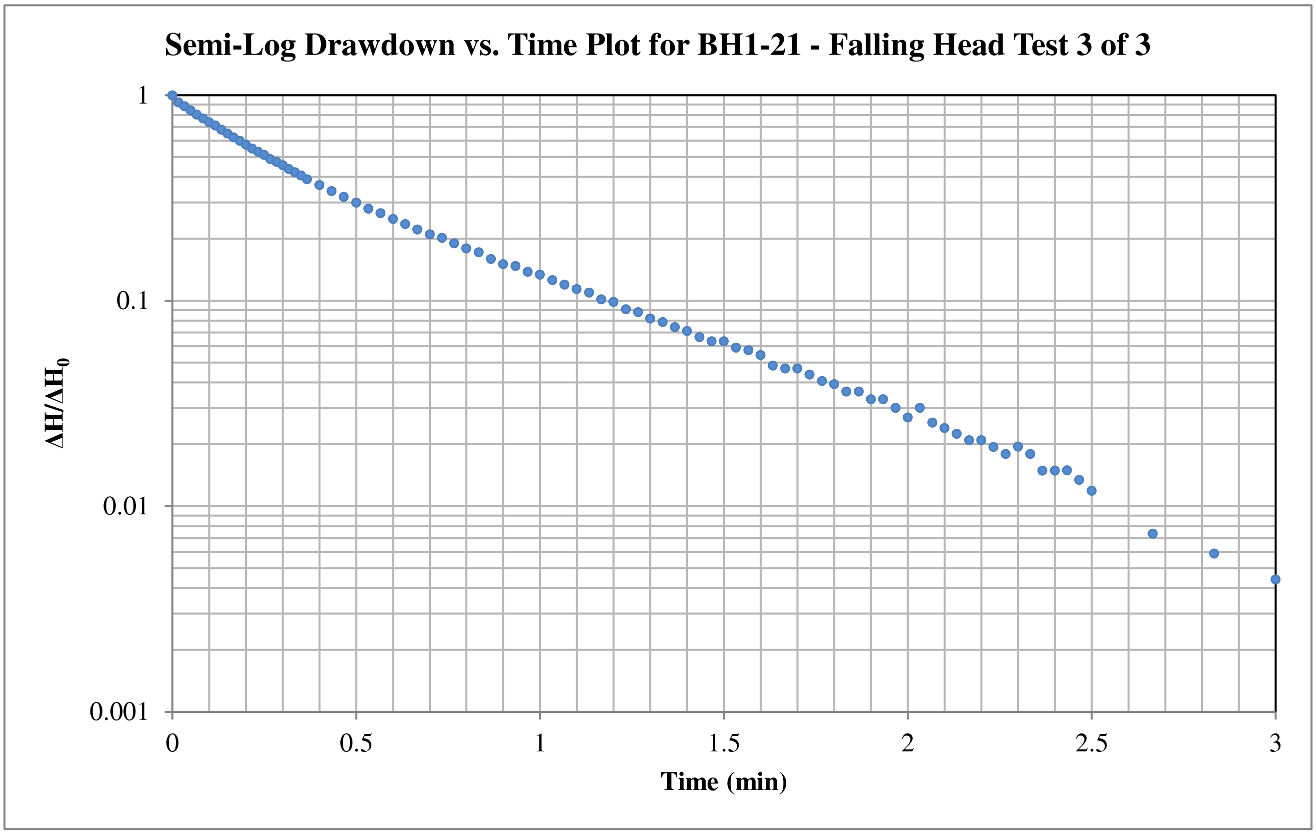
Data Points (from plot):

t^*: 0.341 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.51E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH1-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3 m	Saturated length of screen or open hole
D	0.051 m	Diameter of well
r_c	0.0255 m	Radius of well

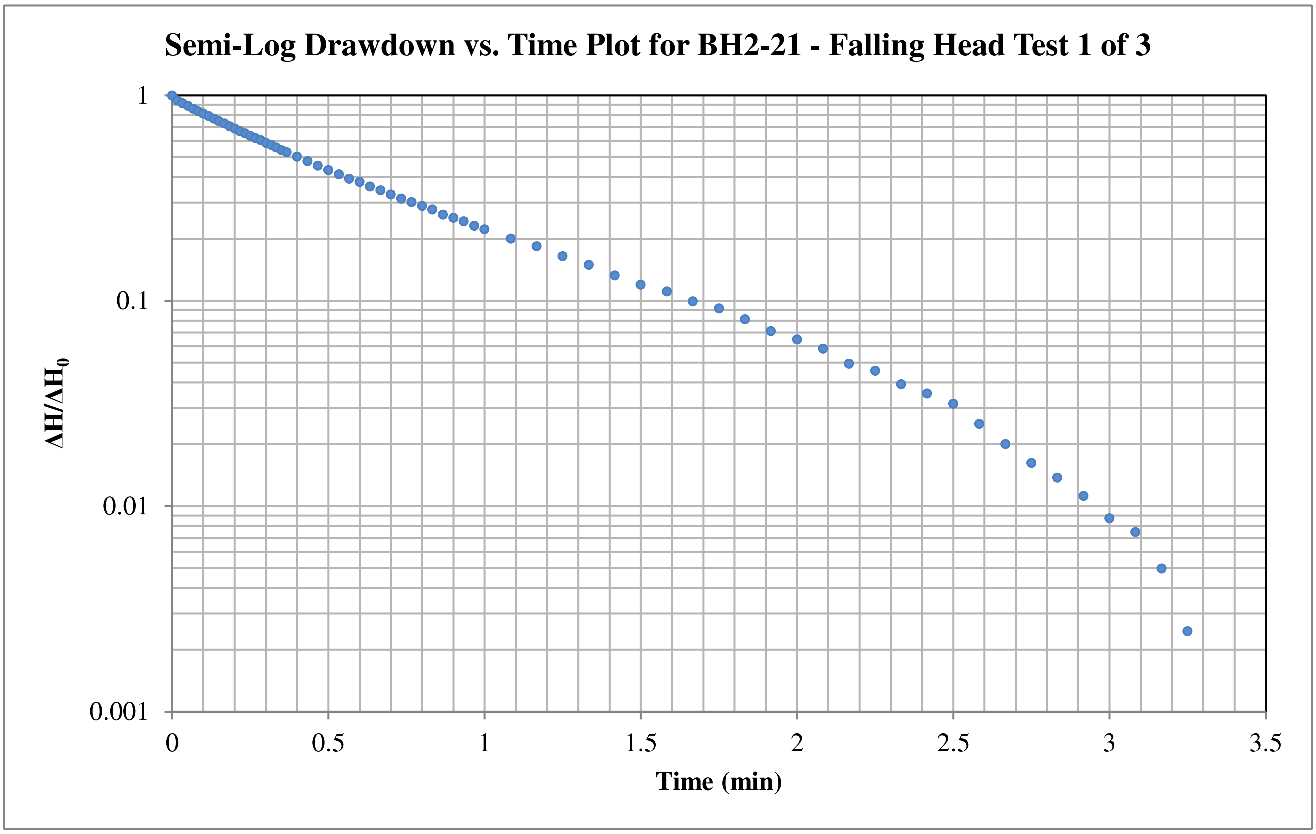
Data Points (from plot):

t^*: 0.395 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.17E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH2-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

Data Points (from plot):

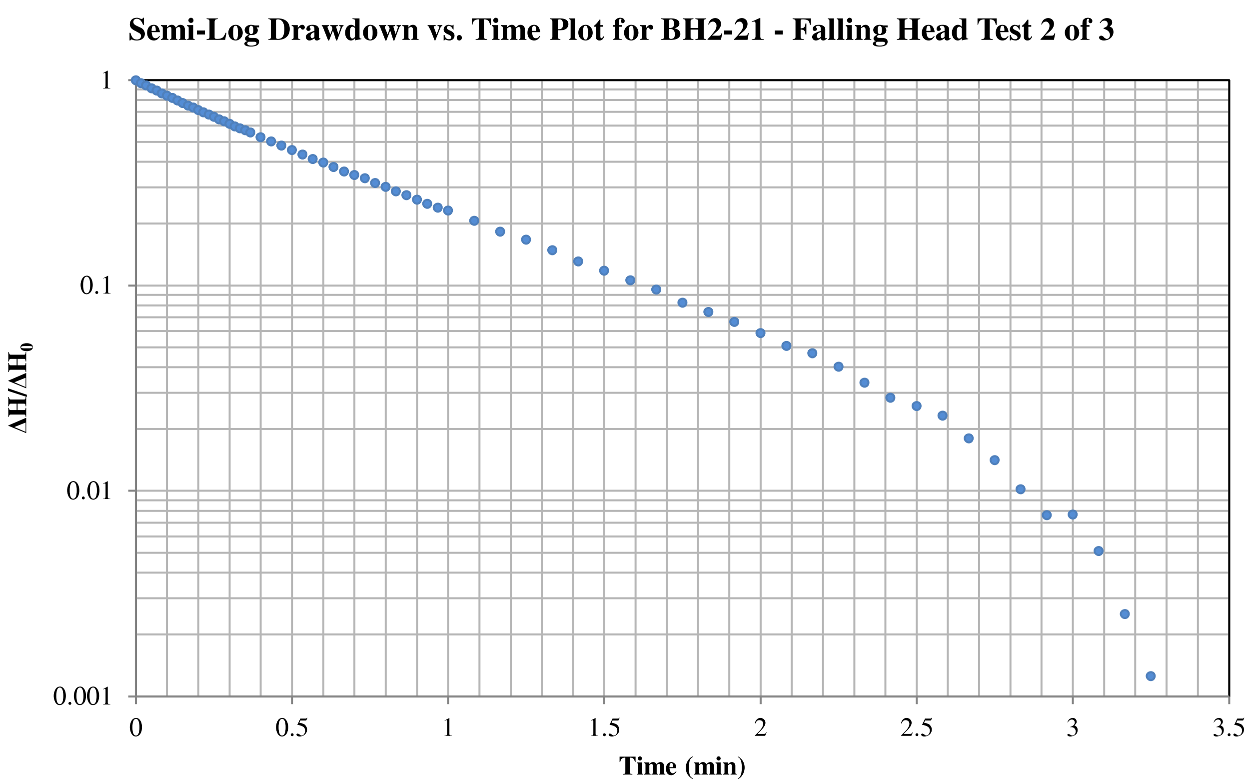
t^*: 0.615 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.39E-05 m/sec

patersongroup

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH2-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

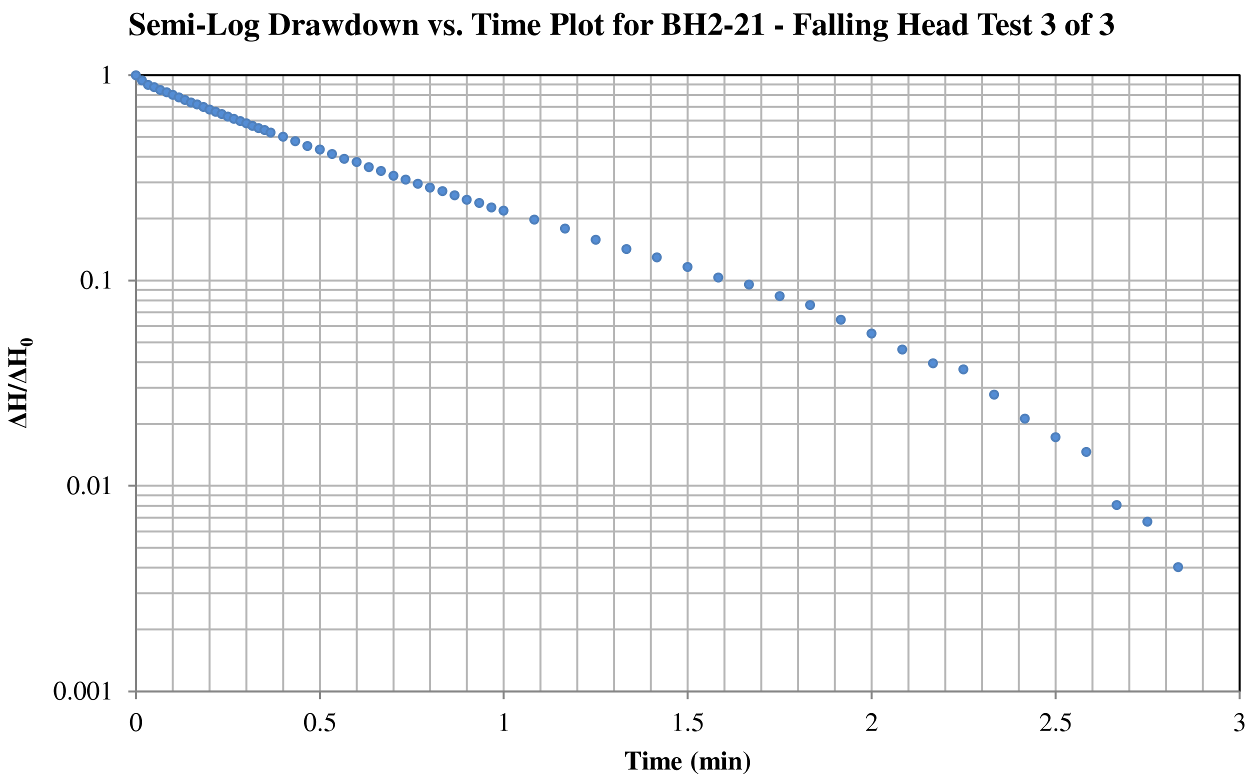
Data Points (from plot):

t^*: 0.646 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.33E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH2-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

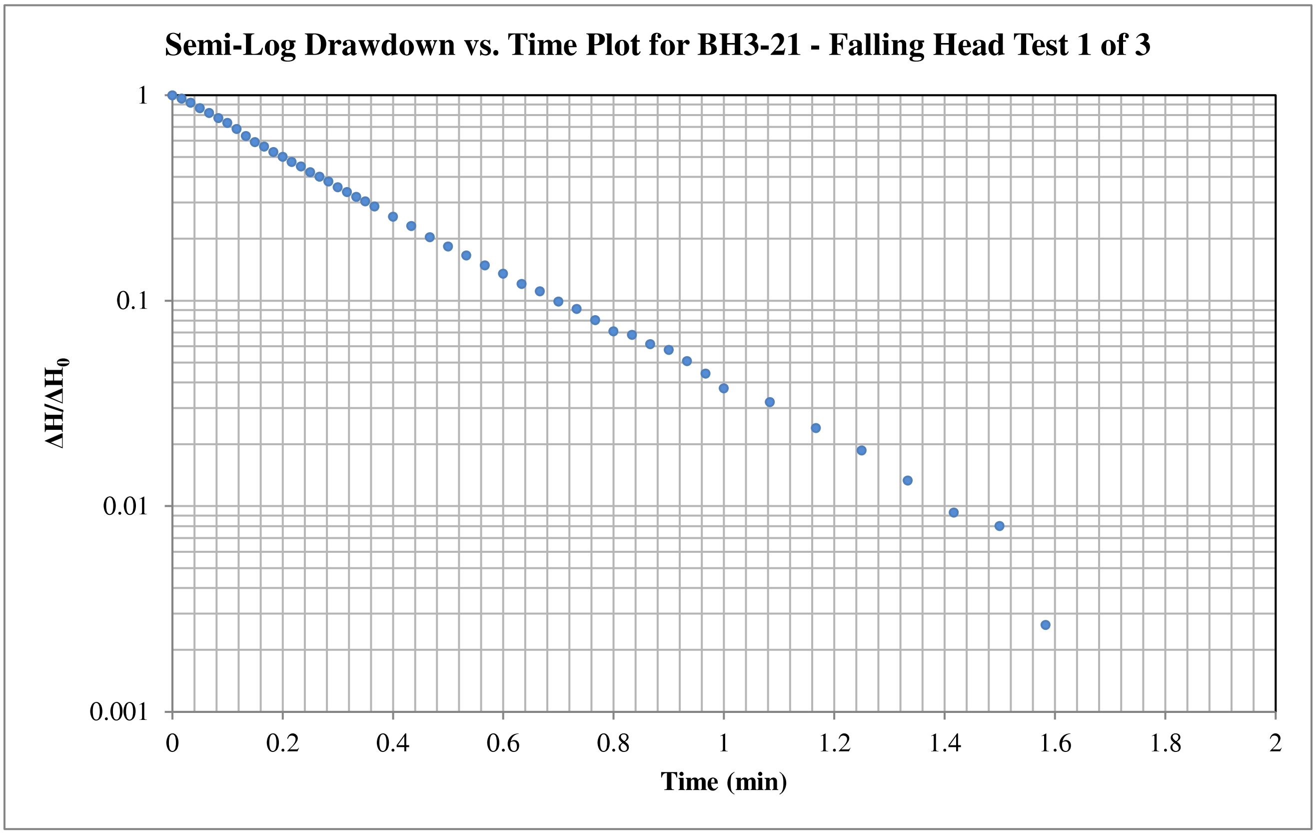
Data Points (from plot):

t^*: 0.608 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.41E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH3-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

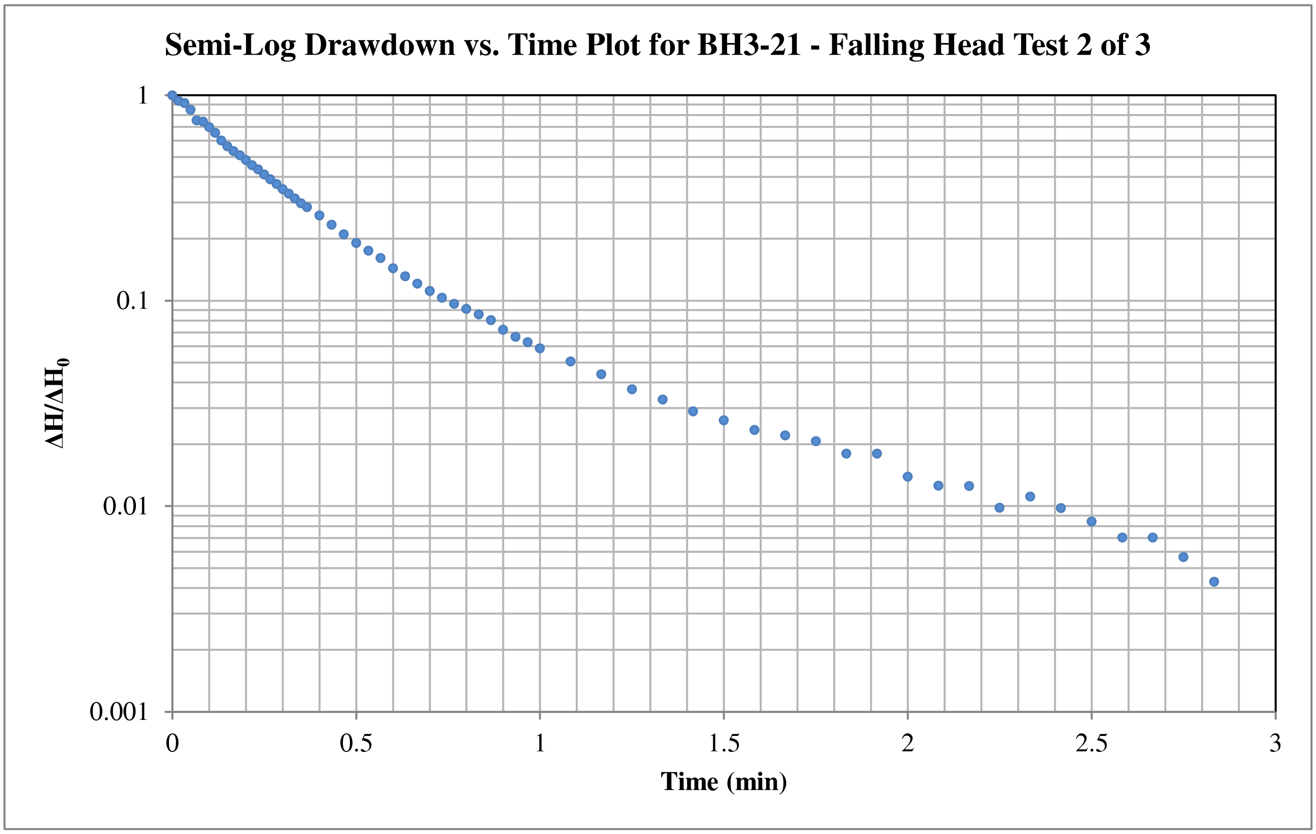
Data Points (from plot):

t^*: 0.291 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.94E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH3-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

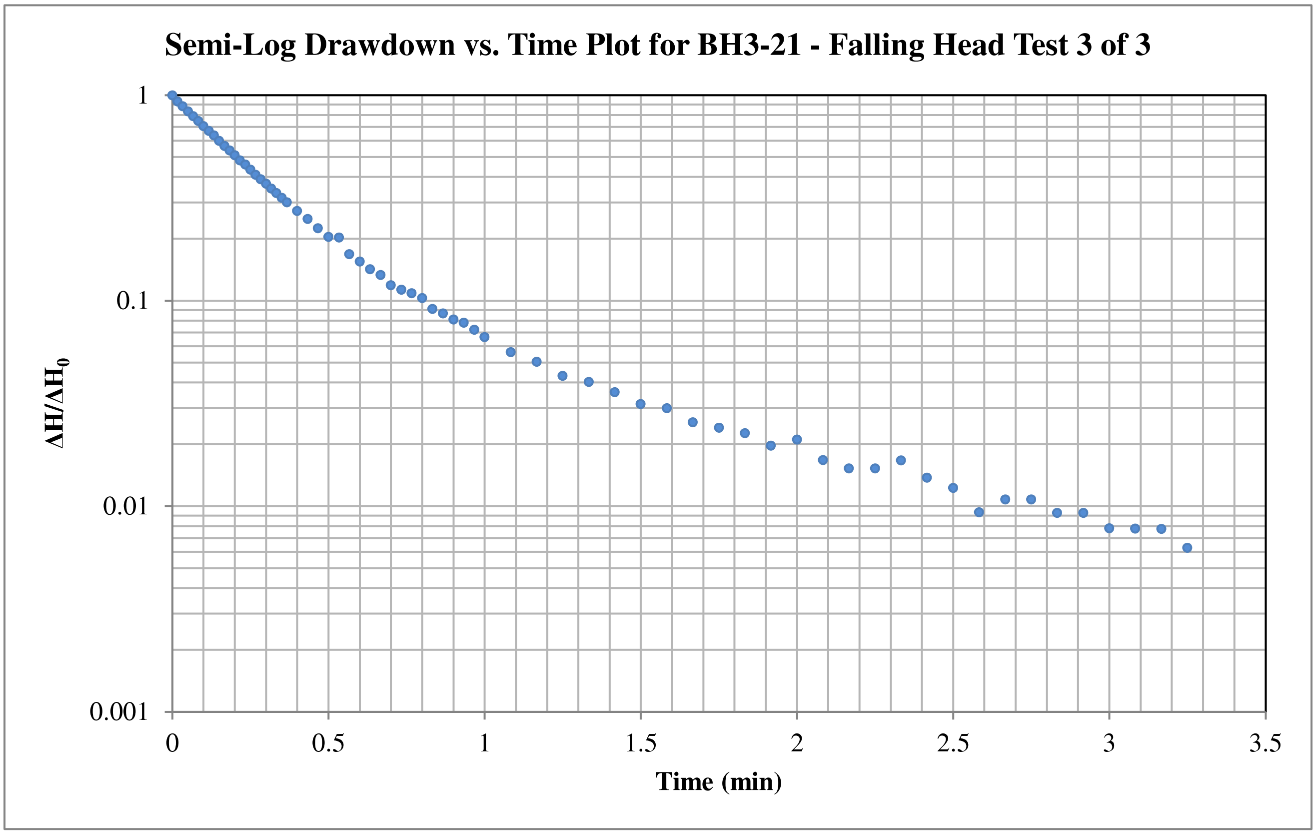
Data Points (from plot):

t^*: 0.283 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 3.02E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH3-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

Data Points (from plot):

t^*: 0.301 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.85E-05 m/sec

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 – AERIAL PHOTOGRAPH – 2008

FIGURE 3 – AERIAL PHOTOGRAPH – 2011

FIGURE 4 – AERIAL PHOTOGRAPH – 2014

FIGURES 5 AND 6 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG7871-1 – TEST HOLE LOCATION PLAN

DRAWING PG7871-2 – PERMISSIBLE GRADE RAISE PLAN

DRAWING PG7871-3 – TREE PLANTING SETBACK PLAN

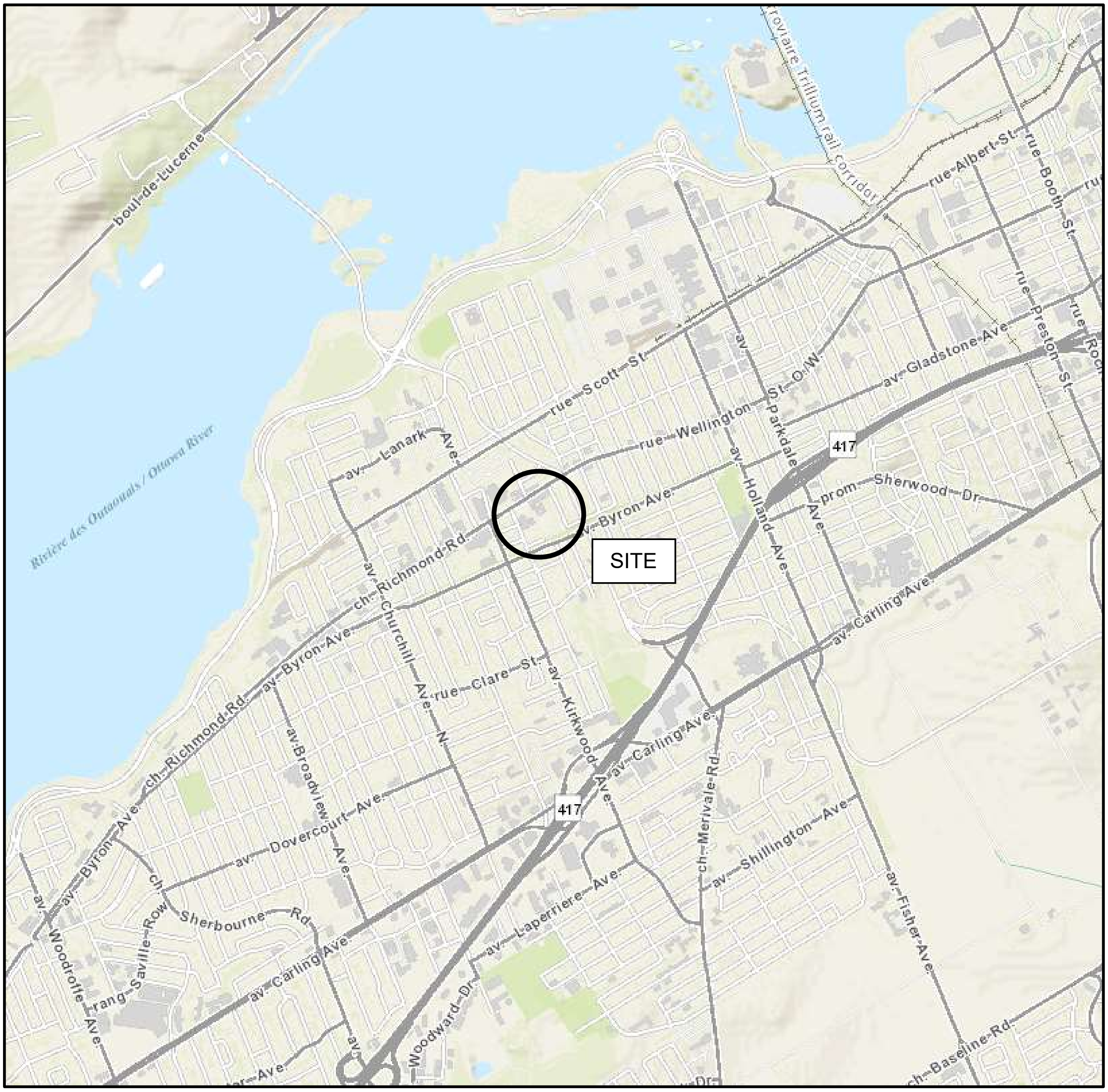


FIGURE 1

KEY PLAN

PATERSON GROUP

FIGURE 2

AERIAL PHOTOGRAPH - 2008

PATERSON GROUP

FIGURE 3

AERIAL PHOTOGRAPH - 2011

PATERSON GROUP

FIGURE 4

AERIAL PHOTOGRAPH - 2014

PATERSON GROUP

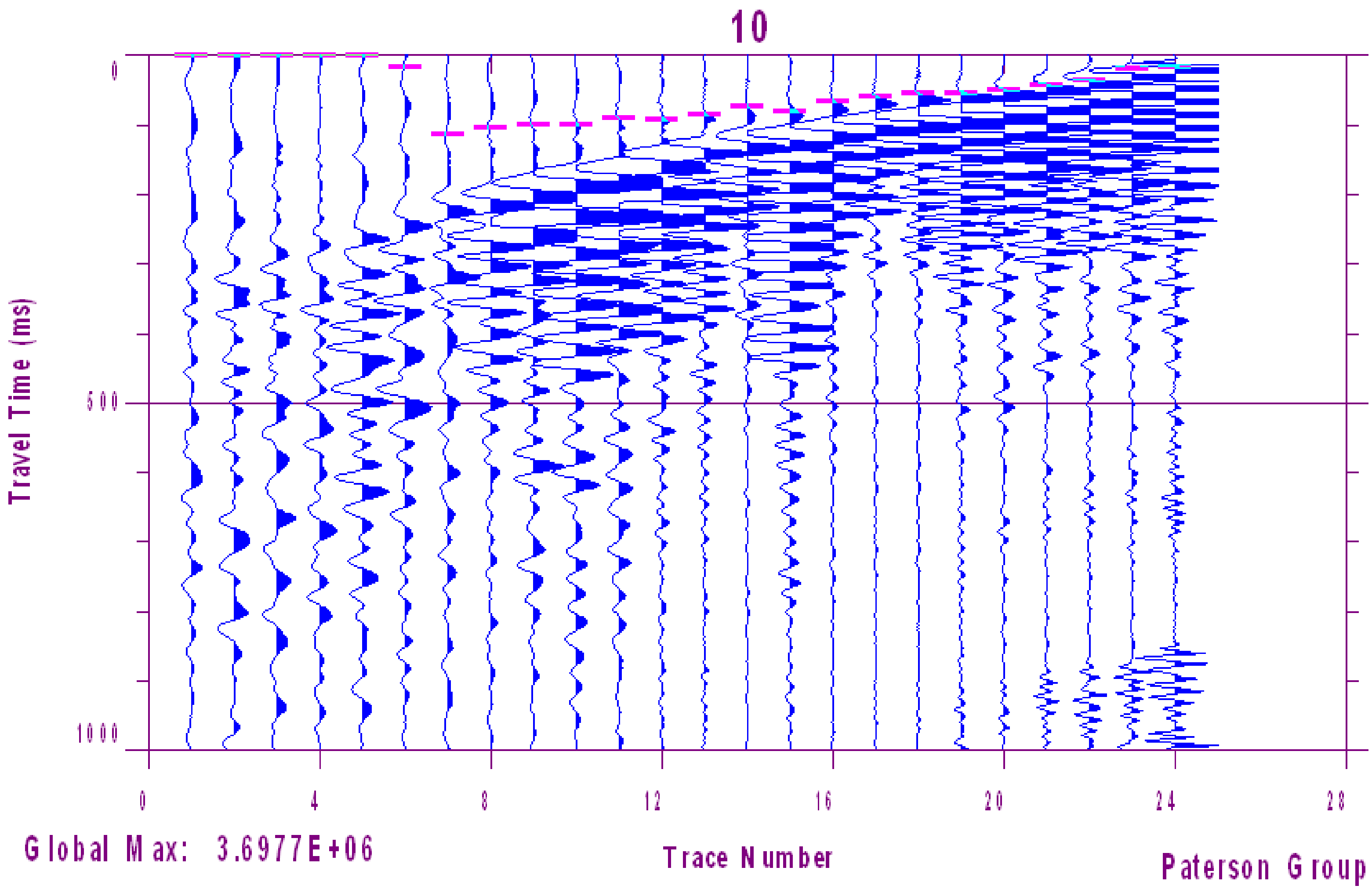


Figure 5 – Shear Wave Profile at Shot Location 72.0 m

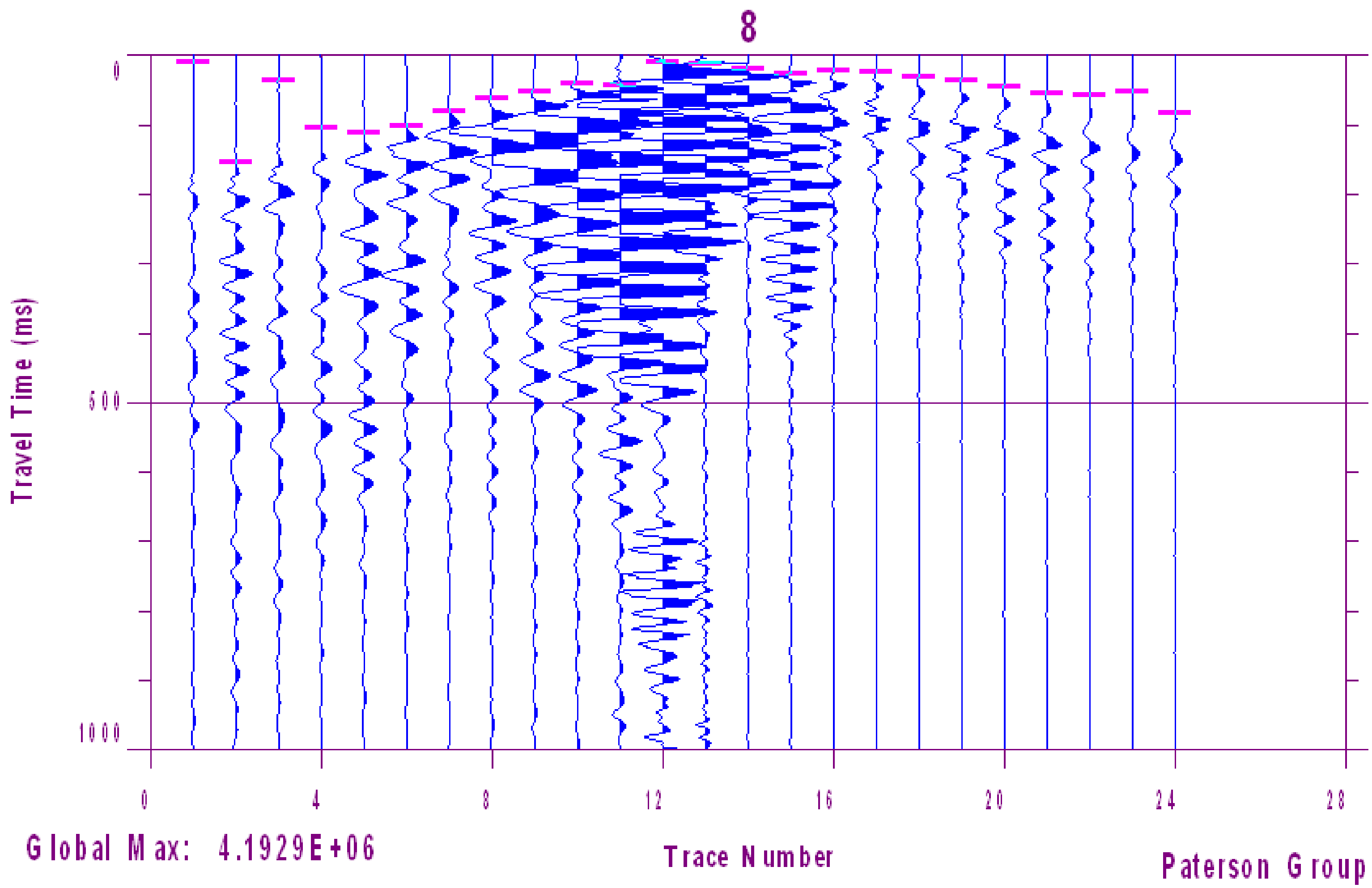
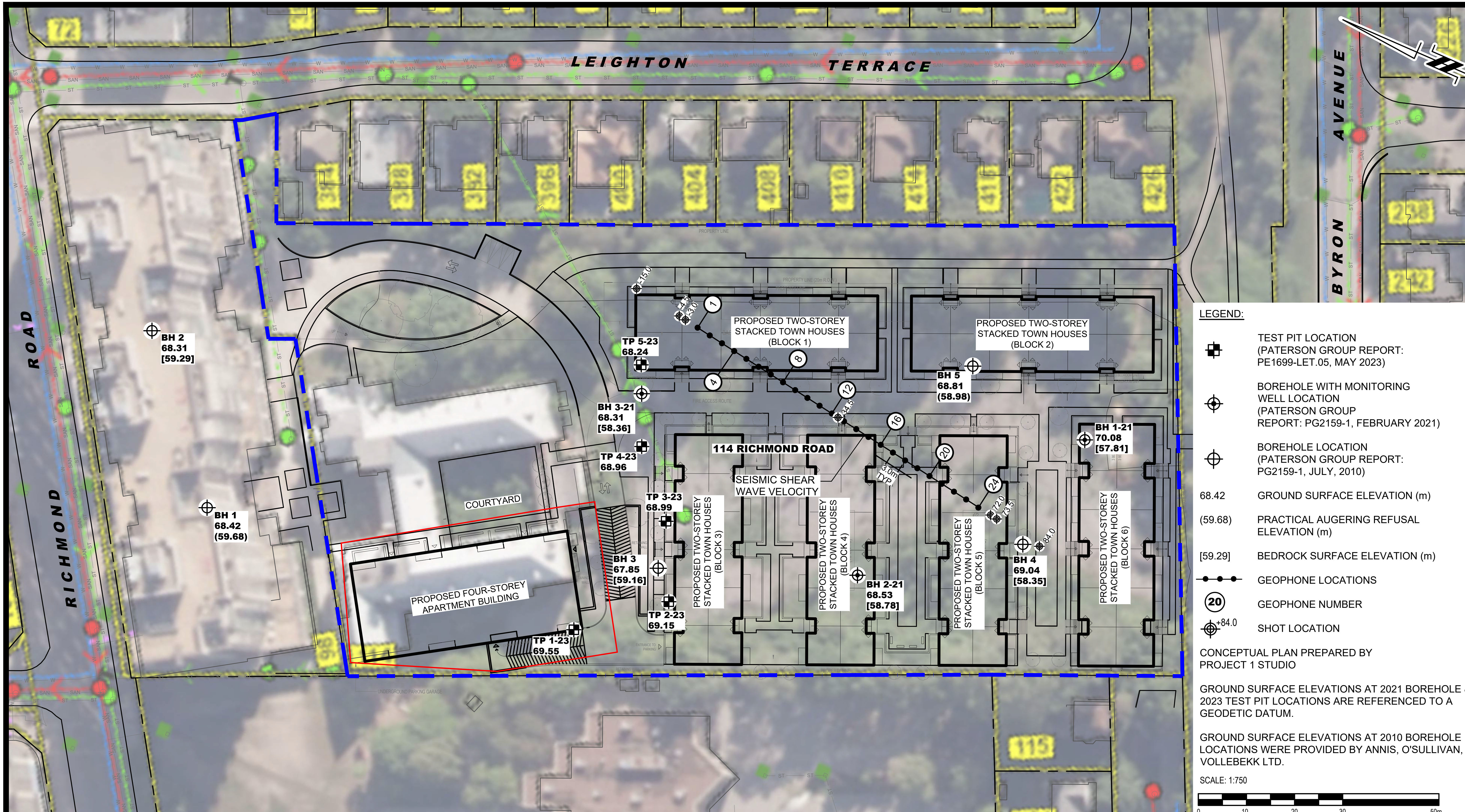


Figure 6 – Shear Wave Profile at Shot Location 34.5 m



PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

CONCORDE PROPERTIES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
114 RICHMOND ROAD

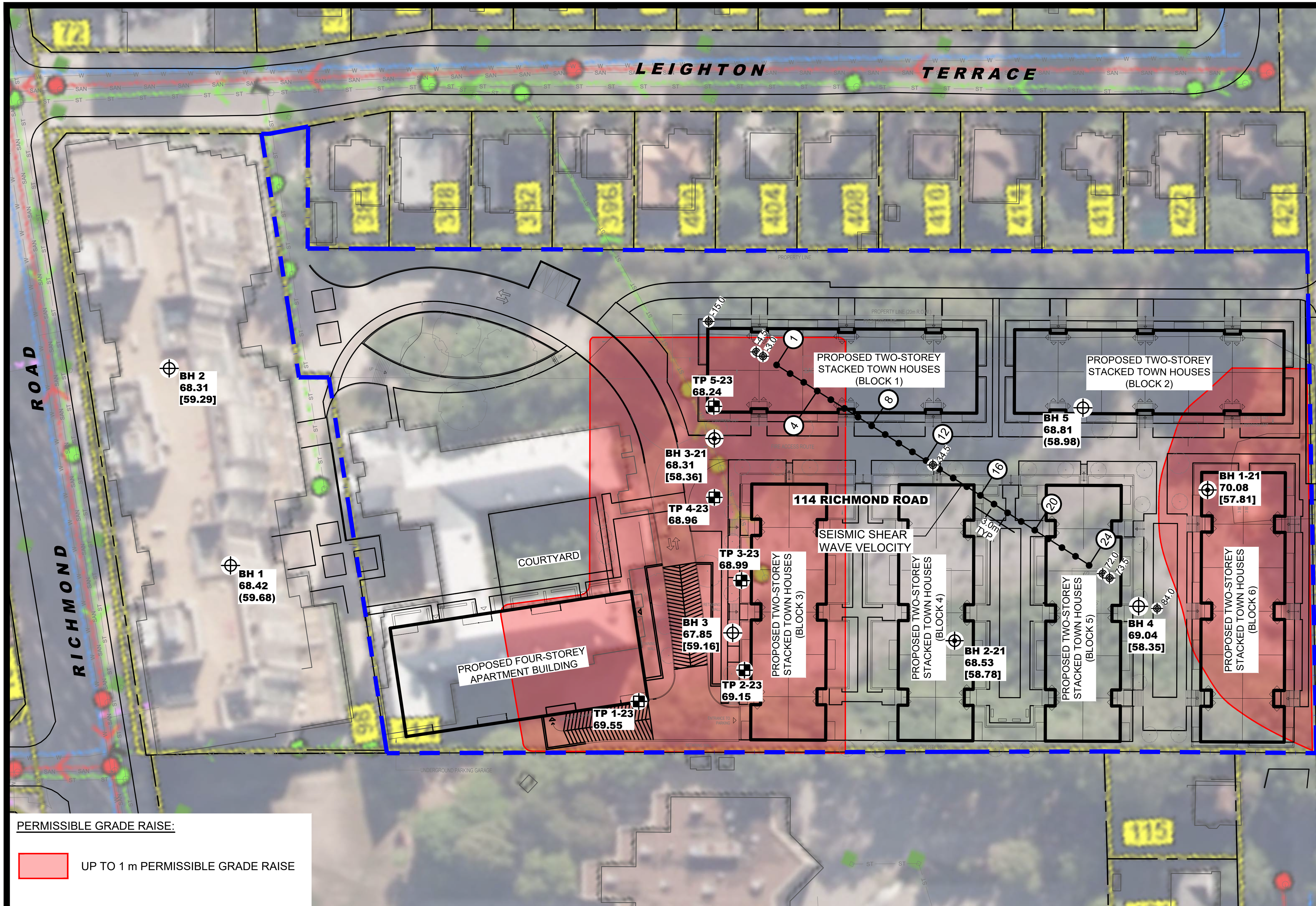
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 02/2026
Drawn by: YA	Report No.: PG7871-1
Checked by: MA	Dwg. No.: PG7871-1
Approved by: KP	Revision No.:

p:\autocad drawings\geotechnical\pg78xx\pg7871\pg7871-1-test hole location plan.dwg

11x17



PERMISSIBLE GRADE RAISE:

UP TO 1 m PERMISSIBLE GRADE RAISE

LEGEND:

TEST PIT LOCATION (PATERSON GROUP REPORT: PE1699-LET.05, MAY 2023)

BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT: PG2159-1, FEBRUARY 2021)

BOREHOLE LOCATION (PATERSON GROUP REPORT: PG2159-1, JULY, 2010)

68.42 GROUND SURFACE ELEVATION (m)

(59.68) PRACTICAL AUGERING REFUSAL ELEVATION (m)

[59.29] BEDROCK SURFACE ELEVATION (m)

GEOPHONE LOCATIONS

20 GEOPHONE NUMBER

+84.0 SHOT LOCATION

CONCEPTUAL PLAN PREPARED BY PROJECT 1 STUDIO

GROUND SURFACE ELEVATIONS AT 2021 BOREHOLE & 2023 TEST PIT LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

GROUND SURFACE ELEVATIONS AT 2010 BOREHOLE LOCATIONS WERE PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

SCALE: 1:750

0 10 20 30 50m

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

CONCORDE PROPERTIES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
114 RICHMOND ROAD

OTTAWA, ONTARIO

Title:

PERMISSIBLE GRADE RAISE PLAN

Scale: 1:750	Date: 02/2026
Drawn by: YA	Report No.: PG7871-1
Checked by: MA	Dwg. No.: PG7871-2
Approved by: KP	Revision No.:

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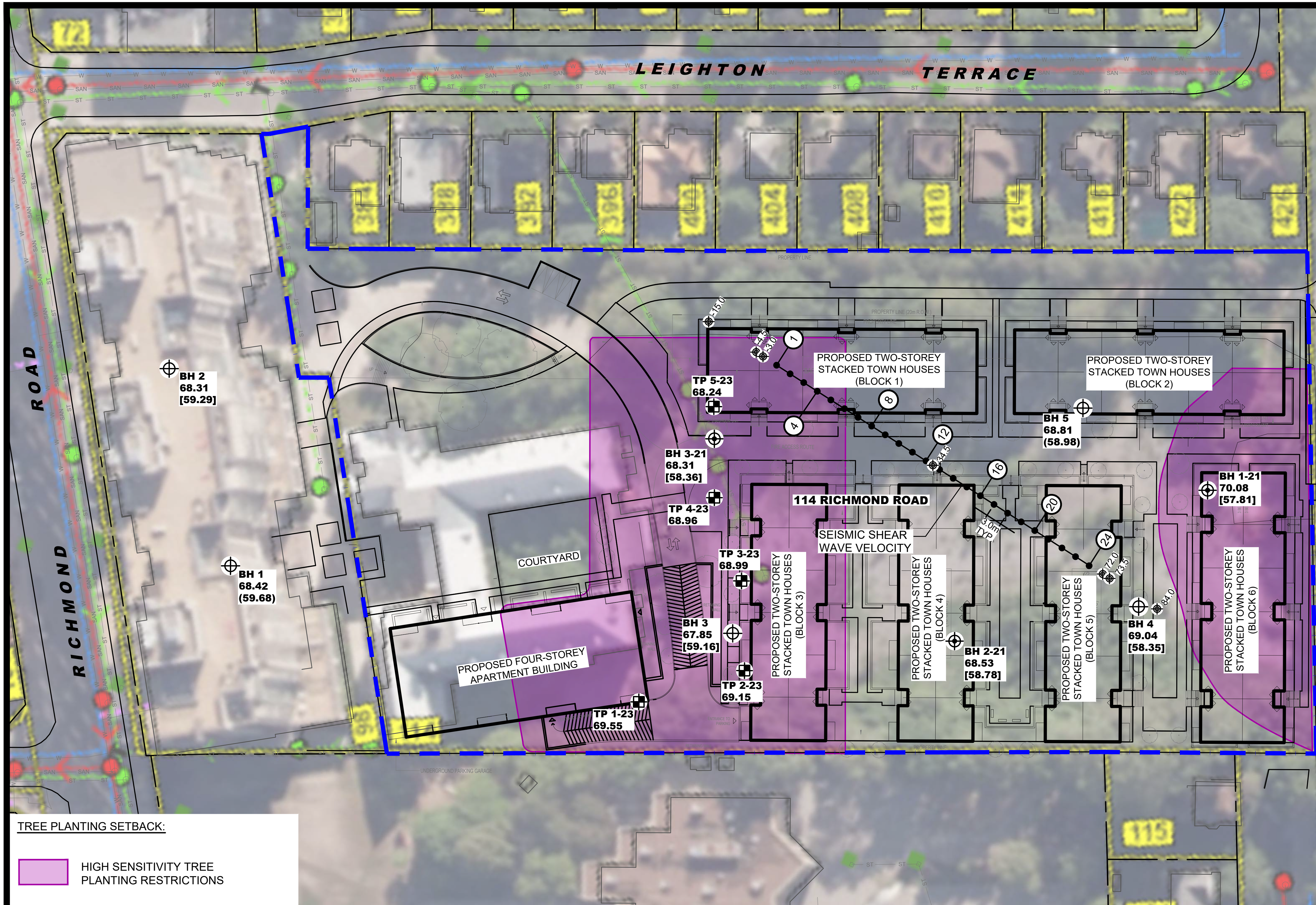
11x17



TREE PLANTING SETBACK:

HIGH SENSITIVITY TREE PLANTING RESTRICTIONS

LEGEND:

TEST PIT LOCATION (PATERSON GROUP REPORT: PE1699-LET.05, MAY 2023)

BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT: PG2159-1, FEBRUARY 2021)

BOREHOLE LOCATION (PATERSON GROUP REPORT: PG2159-1, JULY, 2010)

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GEOPHONE LOCATIONS

20 GEOPHONE NUMBER

+84.0 SHOT LOCATION

CONCEPTUAL PLAN PREPARED BY PROJECT 1 STUDIO

GROUND SURFACE ELEVATIONS AT 2021 BOREHOLE & 2023 TEST PIT LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

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SCALE: 1:750

0 10 20 30 50m

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GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
114 RICHMOND ROAD

OTTAWA, ONTARIO

Title:

TREE PLANTING SETBACK PLAN

Scale: 1:750	Date: 02/2026
Drawn by: YA	Report No.: PG7871-1
Checked by: MA	Dwg. No.: PG7871-3
Approved by: KP	Revision No.:

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