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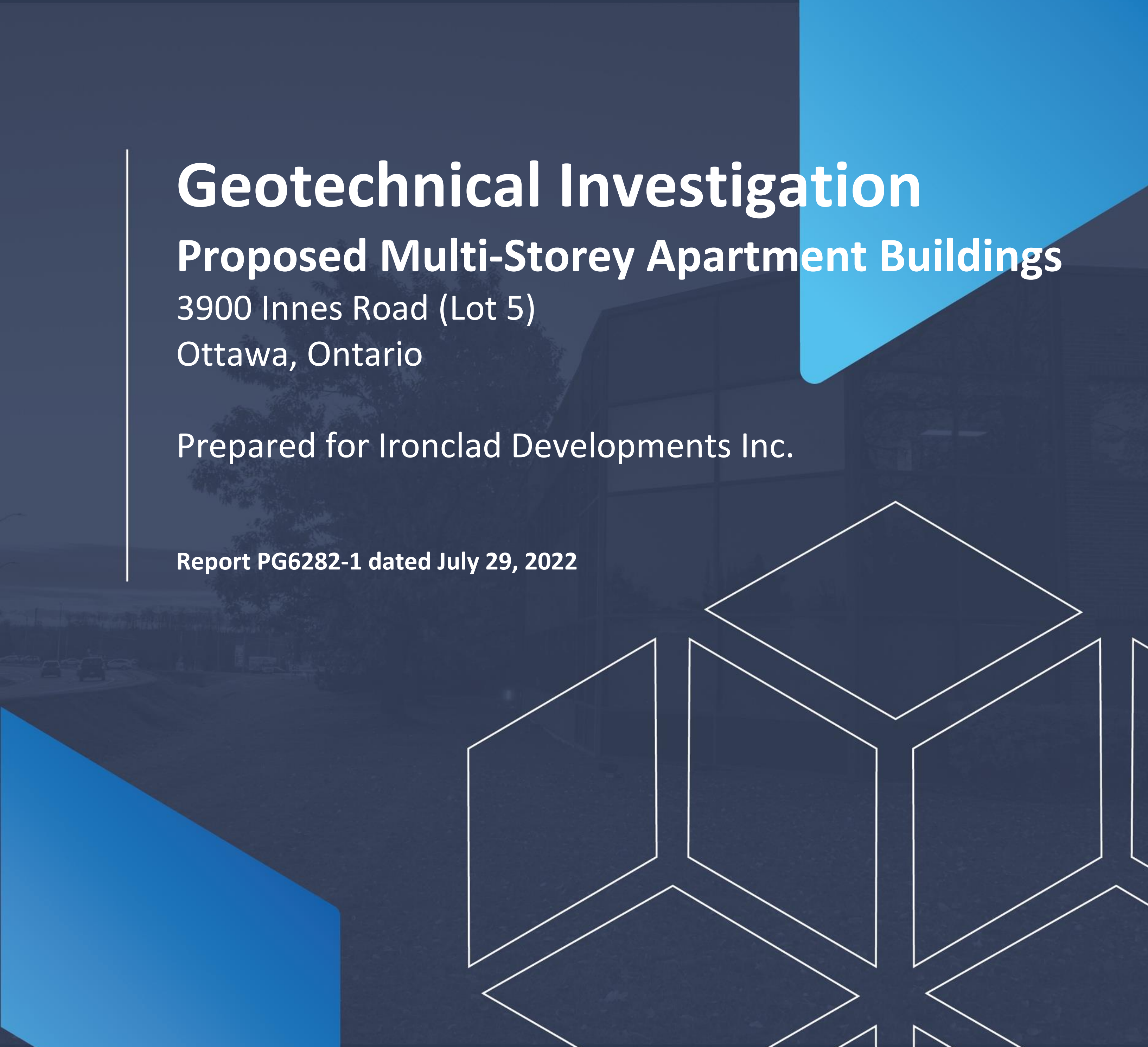

Geotechnical Investigation

Proposed Multi-Storey Apartment Buildings

3900 Innes Road (Lot 5)
Ottawa, Ontario

Prepared for Ironclad Developments Inc.

Report PG6282-1 dated July 29, 2022

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Ironclad Developments Inc. to conduct a geotechnical investigation for the proposed multi-storey apartment buildings (subject site) to be located at Lot 5 within 3900 Innes Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

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

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

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

It is understood the proposed development will consist of two six-storey timber-framed apartment buildings each located above a single level underground basement parking structure. The surface parking areas located centrally throughout the site are not anticipated to be located above an underlying underground parking structure.

The development is anticipated to be provided with landscaped areas, access roads, loading areas and is anticipated to be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on July 7, 2022 and consisted of advancing four (4) boreholes to a maximum depth of 9.2 m and four (4) probe holes to a maximum depth of 2.8 m below the existing ground surface. A previous investigation was undertaken by Paterson in April of 2006. At that time, one borehole and two test pits were advanced to maximum depths of 0.8 m and 2.5 m, respectively.

The test hole locations from the current investigation were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The test hole locations are shown on Drawing PG6282-1 - Test Hole Location Plan included in Appendix 2.

Boreholes and probeholes were advanced using a track-mounted drill rig operated by a two-person crew. The drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden soils. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department.

Sampling and In Situ Testing

Soil samples were recovered from the auger flights, using a 50 mm diameter split-spoon sampler, or core recovery barrels. The split-spoon, auger, and rock core samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory for further examination. The depths at which the split-spoon, auger flights, and rock core samples were recovered from the boreholes are shown as SS, AU, 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 each 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.

Diamond drilling was completed at borehole BH 1-22 to confirm the bedrock quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio of the bedrock sample length recovered over the drilled section length, in percentage.

The RQD value is the total length ratio of intact rock core length more than 100 mm in one drilled section over the length of the drilled section, in percentage. These values are indicative of the quality of the bedrock.

Undrained shear strength testing was carried out at regular depth intervals in cohesive soils using a vane apparatus.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Monitoring wells were installed at boreholes BH 1-22 and BH 2-22, and flexible polyethylene standpipes were installed at boreholes BH 3-22 and BH 4-22 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- Slotted 32 mm diameter PVC screen at the base of each borehole.
- 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- No.3 silica sand backfill within annular space around screen.
- Bentonite hole plug directly above PVC slotted screen.
- Clean backfill from top of bentonite plug to the ground surface.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

3.2 Field Survey

The test hole locations were selected by Paterson personnel in a manner to provide general coverage of the proposed development, taking into consideration existing site features. The ground surface elevations were referenced to a geodetic datum. The test hole locations and ground surface elevations at the test hole locations are presented on Drawing PG6282-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures by Paterson. 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 Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site consists of vacant agricultural land. The ground surface across the site is relatively flat with the exception of an approximately 0.5 m deep ditch crossing in a north-south direction.

The site is bordered to the south by a commercial driving range and to the east, west, and north by vacant agricultural land. The subject site is part of a larger parcel of land which is further bordered by several car-dealerships to the west and a commercial plaza to the north.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of a thin layer of topsoil underlain by a deposit of silty sand and/or silty clay. The clay layer was observed to be underlain by a layer of glacial till, which was further underlain by the bedrock formation.

The silty sand deposit observed at BH 1-22 and BH 2-22 generally consisted of a compact, brown silty sand with clay and trace gravel. An inferred layer of silty sand was observed at PH 2-22 to a depth of approximately 2.3 m below ground surface.

The clay deposit was observed to consist of a hard to stiff, brown silty clay which extended up to a depth of 3.0 m below ground surface.

The glacial till deposit generally consisted of dense, brown silty sand with clay, gravel, cobbles and boulders. The glacial till deposit was observed to extend up to a depth of 5.2 m below ground surface.

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

Bedrock

Bedrock was cored in BH 1-22 to a depth of 9.2 m. The recorded average RQD value ranged from 72 to 100, while the recovery values equaled 100 % at all bedrock depth intervals. Based on these results the quality of the bedrock ranges from good to excellent.

Based on available geological mapping, the bedrock in the subject area consists of interbedded limestone and shale of the Paleozoic formation, as well as interbedded limestone and dolomite of the Gull River formation between the south and north portions of the site, respectively. The anticipated overburden drift thickness ranges between 5 and 15 m depth. Reference can be made to Drawing PG6282-2 – Bedrock Contour Plan for the approximate test hole locations, refusal elevations and approximate bedrock contours based on refusal elevations.

4.3 Groundwater

Groundwater levels were recorded at each borehole location instrumented with a monitoring device. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1. The measured groundwater levels by Paterson are presented in Table 1 below:

Table 1 – Summary of Groundwater Levels

Borehole Number	Observation Method	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
			Depth (m)	Elevation (m)	
BH 1-22	Monitoring Well	88.59	2.27	86.32	July 14, 2022
BH 2-22	Monitoring Well	89.12	Dry	N/A	July 14, 2022
BH 3-22	Piezometer	88.56	2.45	86.11	July 14, 2022
BH 4-22	Piezometer	89.55	Dry	N/A	July 14, 2022

Note: The ground surface elevation at each borehole location was surveyed using a high precision GPS and referenced to a geodetic datum.

Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected to be below the bedrock surface throughout the northern portion of the site where the bedrock surface is within 2 m from ground surface. The groundwater table is expected to be perched within the clay deposit at a depth of approximately **3 to 4 m** throughout the southern portion of the site where the overburden is greater than approximately 3 m.

It should be noted that 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 suitable for the proposed development. The proposed multi-storey apartment buildings are expected to be founded on conventional footings placed on clean, surface sounded bedrock.

Bedrock removal will be required throughout the subject site. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the relatively shallow bedrock depth at the subject site and the anticipated founding level for the proposed buildings, all existing overburden material will be excavated from within the proposed building footprints. Bedrock removal will be required for the construction of the parking garage level.

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

Bedrock Removal

Based on the bedrock encountered in the area, it is expected that line-drilling in conjunction with hoe-ramming and controlled blasting will be required to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

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

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

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

Vibration Considerations

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

The following construction equipment could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of a shoring system with soldier piles or sheet piling will require these pieces of equipment. Vibrations, caused by blasting or construction operations could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the recommended vibration limit, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards. Considering there are several sensitive buildings in close proximity to the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed buildings.

Bedrock Excavation Face Reinforcement

Horizontal rock anchors, shotcrete and/or chain link fencing connected to the excavation face may be required at specific locations to prevent bedrock pop-outs, especially in areas where bedrock fractures are conducive to the failure of the bedrock surface. The requirement for bedrock excavation face reinforcement will be evaluated during the excavation operations.

Overbreak in Bedrock

Sedimentary bedrock formation, such as limestone, dolomite and shale, contain bedding planes, joints and fractures, and mud seams which create natural planes of weakness within the rock mass. Although several factors of a blast may be controlled to reduce backbreak and overbreak, upon blasting, the rock mass will tend to break along natural planes of weakness that may be present beyond the designed blast profile. However, estimating the exact amount of backbreak and overbreak that may occur is not possible with conventional construction drill and blast methods.

Backbreak should be expected to occur along the perimeter of the building excavation footprint with conventional drill and blast bedrock removal methods. Further, overbreak is expected to occur throughout the lowest lifts of blasting due to the variable bedding planes and planes of weakness in the in-situ bedrock. It is very difficult to mitigate significant overblasting given the constraints posed by footing geometry and spacing with respect to the zone of influence of blasts and the bedrocks in-situ characteristics.

Depending on the methodology undertaken by the contractor, efforts taken to minimize backbreak and overbreak may add significant time and costs to the excavation operations and is not guaranteed to completely eliminate the potential for backbreak and overbreak. As such, volume estimates of bedrock to be removed may not be reflective of the actual volume of bedrock that may be required to be removed at the time of construction. This may result in additional materials, such as imported fill and concrete, to make up for additional rock loss.

It is recommended that the blasting operations be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

Fill Placement

Fill placed for grading beneath the building footprints should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II or blast rock fill approved by the geotechnical consultant.

The imported fill material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the buildings 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 lifts with a maximum thickness of 300 mm 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 thin lifts to at least 95% of the material's SPMDD.

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 Drain 6000.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm. Where the fill is open graded, a blinding layer of finer granular fill and/or a woven geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements. This can be assessed at the time of construction. Site-generated blast rock fill should be compacted using a suitably sized smooth drum vibratory roller when considered for placement.

Under winter conditions, if snow and ice is present within the blast rock fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. The geotechnical consultant should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized.

5.3 Foundation Design

Bearing Resistance Values

Shallow footings for the apartment buildings and other auxiliary structures placed on a clean, surface sounded bedrock bearing surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**, incorporating a geotechnical resistance factor of 0.5.

Footings placed on a clean, surface-sounded bedrock bearing surface will be subjected to negligible postconstruction total and differential settlements.

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

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance value provided herein will be subjected to negligible potential postconstruction total and differential settlements.

Lean-Concrete In-Filled Trenches

Where bedrock is encountered below the design underside of footing elevation, it is recommended to lower the bearing surface to a suitable bedrock bearing medium by placing the footings on a lean-concrete in-filled trench extending to sound bedrock. This may be accomplished by excavating near-vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (minimum 15 MPa, 28-day compressive) to the design underside of footing level.

Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock. The trench excavation should be at least 300 mm wider than all sides of the footing at the base of the excavation.

The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below 1.5 m depth). Once approved by Paterson, lean concrete can be poured up to the proposed founding elevation.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, an assessment should be completed to determine the water infiltration and stability of the excavation sidewalls extending to the bedrock surface.

It is anticipated water will be perched upon the bedrock and within the overburden. Where water infiltration cannot be controlled using open sumps within the excavation footprints it is recommended to install a well point adjacent to excavation footprints to lower the water table in advance of sub-excavations, if required and as determined at the time of construction.

Frictional Resistance

An unfactored coefficient of friction of 0.7 is considered applicable for the design of concrete footings supported on clean, surface sounded bedrock at this site.

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 bedrock bearing medium when a plane extending down and out from the bottom edges of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

A heavily fractured, weathered bedrock and/or overburden bearing medium will require a lateral support zone of 1H:1V (or flatter).

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed buildings 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.

Field Program

The seismic array testing location was placed as presented in Drawing PG6282-1 - Test Hole Location Plan, attached to the present report. Paterson field personnel placed 24 horizontal 2.4 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a 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 20, 3 and 2 m away from the first and last geophone, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves.

The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile, immediately below the foundation of the buildings. 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.

Based on our testing results, the average overburden shear wave velocity is **360 m/s**, while the bedrock shear wave velocity is **2,062 m/s**. It is understood that the proposed buildings footings will be placed directly on the bedrock surface or indirectly by the use of lean-concrete trenches extended to bedrock surface.

Based on this, the V_{s30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{30\ m}{2{,}062\ m/s}\right)}$$

$$V_{s30=}\ 2{,}062\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30}, for the proposed buildings founded on bedrock is **2,062 m/s**. Therefore, a **Site Class A** is applicable for design of the proposed buildings as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil and deleterious materials within the footprint of the proposed buildings, an approved soil subgrade or bedrock surface, approved by Paterson personnel at the time of construction, is considered to be an acceptable subgrade surface on which to commence backfilling for the floor slab construction.

The recommended pavement structures noted in Subsection 5.7 will be applicable for the founding level of the proposed parking garage structure. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm of clear crushed stone.

An engineered fill such as an OPSS Granular A or Granular B Type II compacted to 98% of its SPMDD could be placed around the proposed footings. Alternatively, excavated bedrock could be used as select subgrade material around the proposed building footings, if well graded with maximum particle size of 300 mm in its longest dimension and approved by the geotechnical consultant at the time of placement.

A subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed. This is discussed further in Section 6.1 of this report.

5.6 Basement Wall

It is understood that the basement walls are to be poured against a dampproofing system, which will be placed against the exposed bedrock face. Below the bedrock surface, a nominal coefficient for at-rest earth pressure of 0.05 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face.

Where the soil is to be retained, there are several combinations of backfill materials and retaining soils for the basement walls of the subject structure. However, the conditions should be designed by assuming the retaining 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 both the static earth pressure component (P_o) and the seismic component ($\triangle P_{AE}$).

Lateral Earth Pressures

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

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

An additional pressure having a magnitude equal to $Ko \cdot q$ 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 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 forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Design

Car only parking and heavy traffic areas are anticipated at this site. The subgrade material will consist of glacial till and bedrock throughout the lowest basement level of the subject site. The subgrade is anticipated to consist of overburden and/or bedrock for surface parking areas. The proposed pavement structures are shown in Tables 2 and 3.

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

Table 3 - Recommended Pavement Structure - Heavy-Truck Traffic and Loading Areas

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soils, bedrock or OPSS Granular B Type I or II material placed over in situ soil or bedrock.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

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

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing load carrying capacity.

Due to the low permeability of the subgrade materials consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. It is expected that insufficient room will 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 drainage system consist of the following:

❏ A composite drainage membrane (DeltaDrain 6000, MiraDrain G100N or equivalent) should be placed against the shoring system and bedrock excavation face from the finished ground surface to the top of the footing. The bedrock face is recommended to be grinded to provide a smooth-surface for the installation of the drainage board layer. Large cavities should be reviewed by the geotechnical consultant to assess the requirement to in-fill cavities suitably to facilitate the installation of the drainage board layer.

❏ It is recommended that 150 mm diameter PVC sleeves at 3 m centres 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.

❏ 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.

Waterproofing layers for podium deck surfaces should overlap across and below the top endlap of the vertically installed composite foundation drainage board to mitigate the potential for water to perch between the drainage board and foundation wall. Elevators and any other pits located below the underslab drainage system should be waterproofed. Supplemental details, review of architectural design drawings and additional information may be provided by Paterson for these items once the building design has been finalized. It is recommended that Paterson reviews all details associated with the foundation drainage system prior to tender.

Interior Perimeter and Underfloor Drainage

The interior perimeter and underfloor drainage system will be required to control water infiltration below the lowest underground parking level slab and redirect water from the buildings foundation drainage system to the buildings sump pit(s). The interior perimeter and underfloor drainage pipe should consist of a 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock.

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 system should be confirmed by Paterson once the foundation layout and sump system location has been finalized.

Foundation Backfill

Above the bedrock surface, 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 Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

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.

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.

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

However, foundations which are founded directly on clean, surface-sounded bedrock with no cracks or fissures, and which is approved by Paterson at the time of construction, is not considered frost susceptible and does not require soil cover.

Where the bedrock is considered frost susceptible (i.e., weathered bedrock or bedrock with significant fissures filled with soil), foundation insulation will need to be provided. Alternatively, frost susceptible bedrock will need to be removed and replaced with lean concrete (minimum 17 MPa 28-day strength). It is recommended Paterson field personnel review the frost susceptibility of bedrock surface located within 1.8 m of finished grade.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

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 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.

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.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods.

The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team.

Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should 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 design prior to implementation.

The temporary system could consist of 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 included to the earth pressures described below.

These systems could be cantilevered, anchored, or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The 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

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight is calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

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. However, when the bedding is located within bedrock subgrade, a minimum of 300 mm of OPSS Granular A should be placed for bedding for sewer or water pipes. 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 will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement. Well fractured bedrock should be acceptable as backfill for the lower portion of the trenches when the excavation is within bedrock provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones are 300 mm or smaller in their longest dimension.

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

It is anticipated that groundwater infiltration into the excavations through the overburden materials and bedrock 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

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) will be required for this project as it is anticipated that more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP. All water takings under a PTTW are required to be reported to the MECP Water Taking Reporting Systems (WTRS).

Impact on Neighboring Properties

Based on our observations, the long-term groundwater level is expected to be below the founding depth of the proposed structures and groundwater lowering is not anticipated under short-term conditions due to construction of the proposed buildings. The neighboring structures are expected to be founded on bedrock. Issues are not expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed buildings.

6.6 Winter Construction

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

Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions.

In particular, where a shoring system is constructed, 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.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and tarpaulins or other suitable means. 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 buildings and the footings are protected with sufficient soil cover to prevent freezing at founding level.

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

Under winter conditions, if snow and ice is present within the blast rock or other imported fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. The geotechnical consultant should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized in settlement-sensitive areas.

6.7 Corrosion Potential and Sulphate

The results of analytical testing from an adjacent site 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 moderate to slightly aggressive corrosive environment.

7.0 Recommendations

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 the geotechnical consultant:

- Review of the geotechnical aspects of the excavation contractor’s shoring design, prior to construction, if applicable.
- Review the bedrock stabilization and excavation requirements at the time of construction.
- Review of foundation drainage, underfloor drainage and waterproofing details for elevator shafts.
- Review and inspection of the installation of the foundation drainage systems.
- Observation of all bearing surfaces prior to the placement of concrete.
- 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.
- 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 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 Ironclad Developments Inc. 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
July 29, 2022
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B. Eng.

David J. Gilbert, P.Eng.

Report Distribution:

☐ Ironclad Developments Inc. (Digital copy)
☐ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers
9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - Lot 5 - 3900 Innes Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

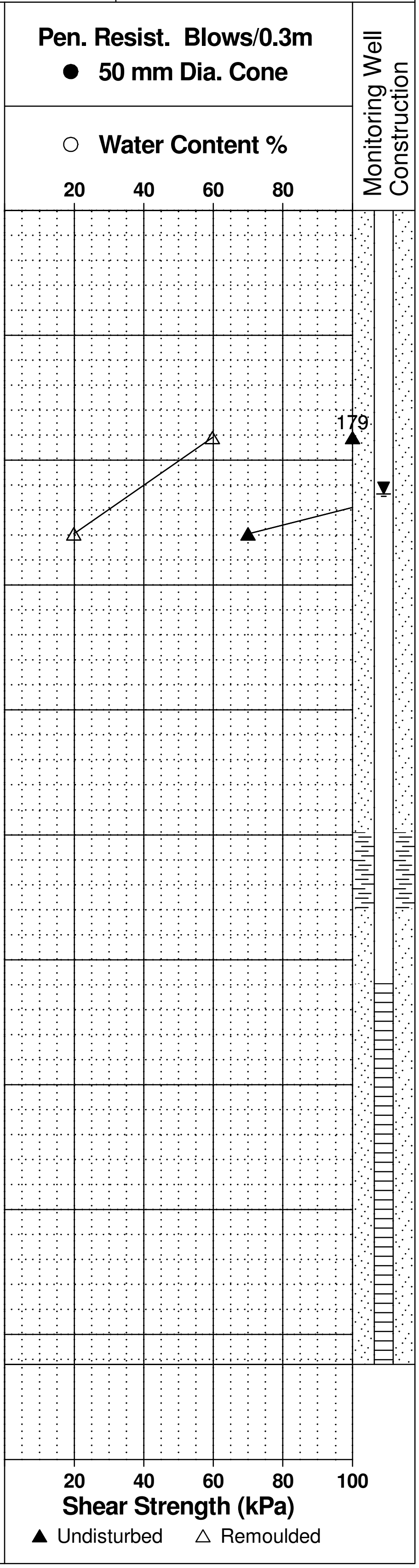
DATE July 7, 2022

FILE NO. PG6282

HOLE NO. BH 1-22

SOIL DESCRIPTION	Depth of base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.25				
Brown **SILTY SAND,** trace clay and gravel	0.69	AU	1		
Very stiff to stiff, brown **SILTY CLAY**	2.97	SS	2	83	9
		SS	3	100	P
		SS	4	100	P
GLACIAL TILL: Compact to dense, brown silty sand with clay, gravel, cobbles and boulders	3.58	SS	5	53	50+
BEDROCK: Good to excellent quality, grey limestone	9.24	RC	1	100	97
		RC	2	100	72
		RC	3	100	100
		RC	4	100	100
End of Borehole					
(GWL @ 2.27m - July 14, 2022)					

DEPTH (m)	ELEV. (m)
0	88.59
1	87.59
2	86.59
3	85.59
4	84.59
5	83.59
6	82.59
7	81.59
8	80.59
9	79.59



patersongroup Consulting Engineers
9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - Lot 5 - 3900 Innes Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE July 7, 2022

FILE NO. PG6282

HOLE NO. BH 2-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Monitoring Well Construction
GROUND SURFACE						0	89.12		
TOPSOIL 0.25									
Loose to compact, brown **SILTY SAND** with clay		AU	1						
1.14		SS	2	79	14	1	88.12		
GLACIAL TILL: Compact, brown silty sand to sandy silt with gravel, cobbles, boulders, trace clay		SS	3	54	24	2	87.12		
2.21									
BEDROCK: Poor to good quality, grey limestone		SS	4	26	34				
2.87									
End of Borehole Practical refusal to augering encountered below bedrock surface at 2.87m depth. (BH dry - July 14, 2022)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - Lot 5 - 3900 Innes Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE July 7, 2022

FILE NO. **PG6282**

HOLE NO. **BH 3-22**

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
TOPSOIL	0.25
Very stiff to stiff, brown **SILTY CLAY**	2.97
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders	5.16

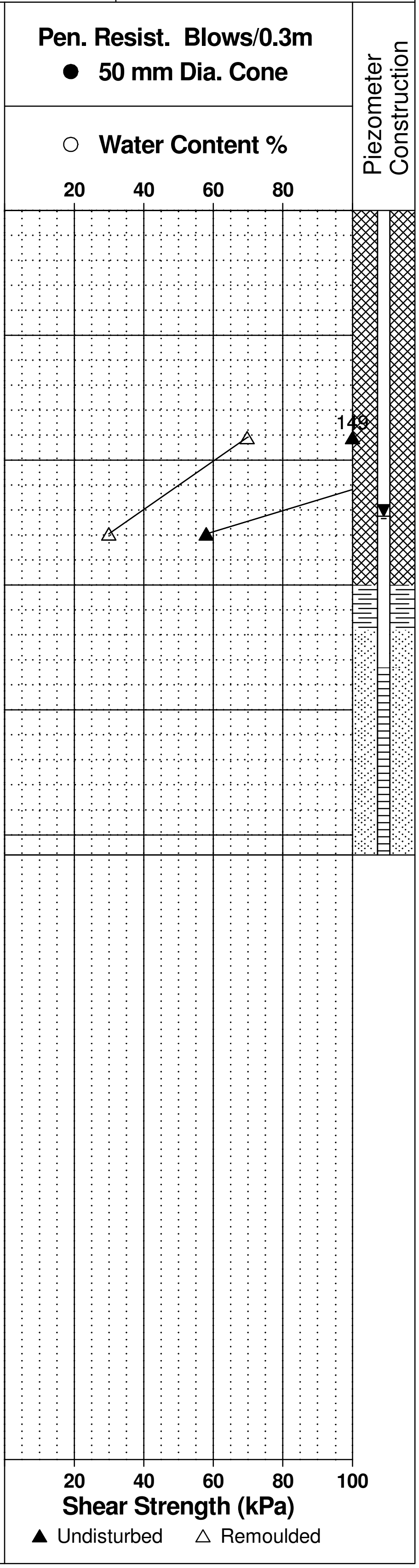
End of Borehole

Practical refusal to augering at 5.16m depth.

(GWL @ 2.45m - July 14, 2022)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	83	7
SS	3	100	P
SS	4	100	P
SS	5	62	4
SS	6	62	13
SS	7	100	18

DEPTH (m)	ELEV. (m)
0	88.56
1	87.56
2	86.56
3	85.56
4	84.56
5	83.56



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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - Lot 5 - 3900 Innes Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE July 7, 2022

FILE NO. PG6282

HOLE NO. BH 4-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.35
TOPSOIL 0.20							
Hard to very stiff, brown **SILTY CLAY** 1.01		AU	1				
GLACIAL TILL: Brown silty clay with sand and gravel 1.19		SS	2	100	14	1	88.35
End of Borehole Practical refusal to augering at 1.19m depth. (BH dry - July 14, 2022)							

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

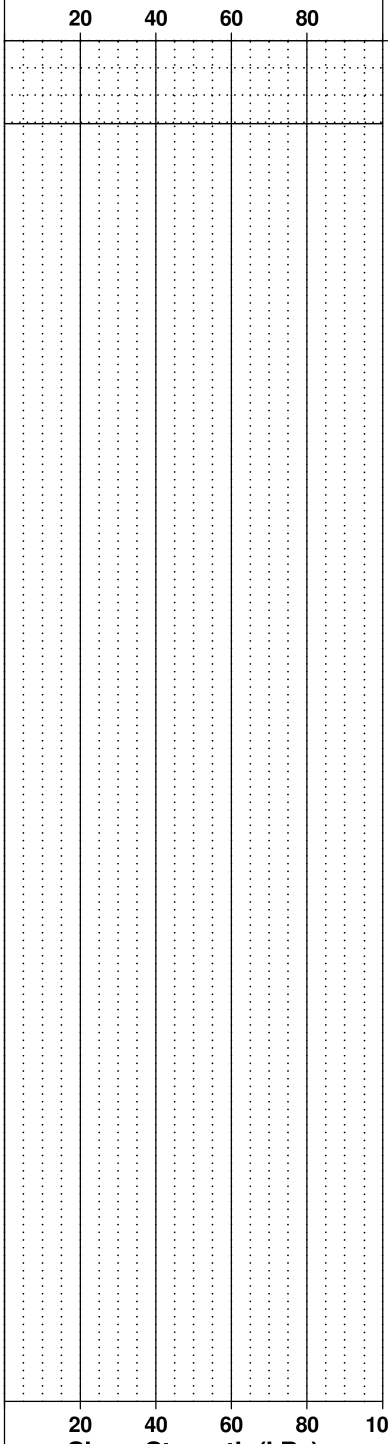
patersongroup Consulting Engineers
9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - Lot 5 - 3900 Innes Road
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Track-Mount Power Auger
DATE July 7, 2022
FILE NO. PG6282
HOLE NO. PH 1-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	89.50		
TOPSOIL 0.25	■								
Brown **SILTY CLAY,** some sand 0.61	▨	AU	1						
End of Probehole Practical refusal to augering at 0.61m depth.									



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - Lot 5 - 3900 Innes Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE July 7, 2022

FILE NO. PG6282

HOLE NO. PH 2-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	89.04		
TOPSOIL 0.25									
Inferred brown **SILTY SAND to SANDY SILT**		AU	1			1	88.04		
						2	87.04		
2.34									
Inferred **GLACIAL TILL** 2.41		AU	2						
End of Probehole Practical refusal to augering at 2.41m depth.									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - Lot 5 - 3900 Innes Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger **DATE** July 7, 2022

FILE NO. PG6282

HOLE NO. PH 3-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	88.59		
TOPSOIL 0.18									
Inferred **SILTY SAND** 0.79		AU	1						
		AU	2			1	87.59		
Inferred **SILTY CLAY**									
		AU	3			2	86.59		
2.82		AU	4						
End of Probehole Practical refusal to augering at 2.82m depth.									

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Buildings - Lot 5 - 3900 Innes Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

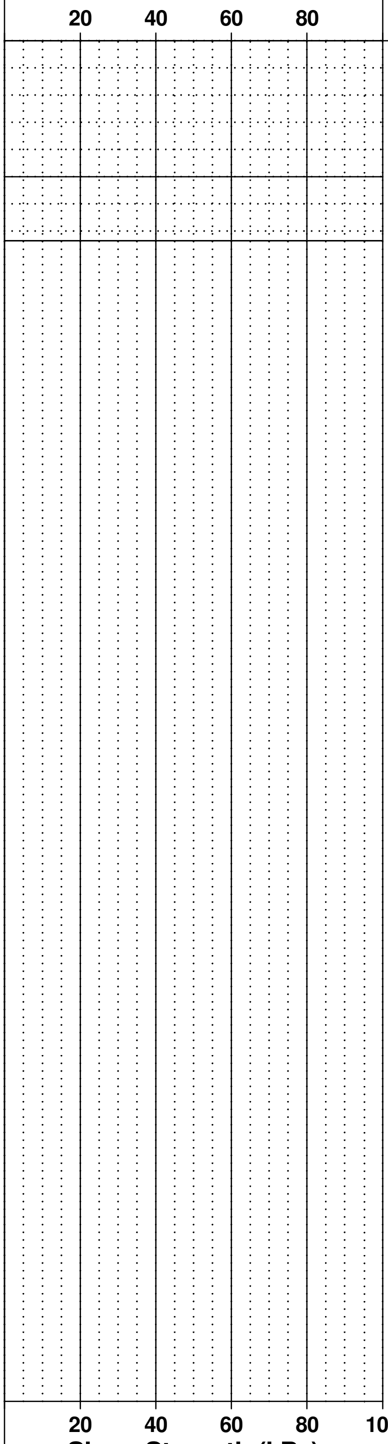
BORINGS BY Track-Mount Power Auger

DATE July 7, 2022

FILE NO. PG6282

HOLE NO. PH 4-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE									
TOPSOIL 0.25						0	88.91		
Inferred **SILTY SAND** 1.47		AU	1			1	87.91		
End of Probehole Practical refusal to augering at 1.47m depth.									



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

SOIL PROFILE & TEST DATA

Preliminary Geotechnical Investigation
Pharand Lands - Innes Road at Mer Bleeu Road
Ottawa, Ontario

DATUM Geodetic, as provided by Stantec Consulting Ltd.

REMARKS

BORINGS BY CME 75 Power Auger

DATE 5 APR 06

FILE NO. PG0811

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0	89.47		
TOPSOIL 0.25	■	AU	1						
Brown SILTY CLAY, trace sand 0.76	▨								
End of Borehole Practical refusal to augering @ 0.76m depth									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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

SOIL PROFILE & TEST DATA

Preliminary Geotechnical Investigation
Pharand Lands - Innes Road at Mer Bleeu Road
Ottawa, Ontario

DATUM Geodetic, as provided by Stantec Consulting Ltd.

FILE NO. PG0811

REMARKS

HOLE NO. TP30

BORINGS BY Backhoe

DATE 12 APR 06

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	89.09		
TOPSOIL 0.20	■								
Stiff, brown SILTY CLAY 1.10	▨					1	88.09		
End of Test Pit TP terminated on bedrock surface @ 1.10m depth (TP dry upon completion)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

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

SOIL PROFILE & TEST DATA

Preliminary Geotechnical Investigation
Pharand Lands - Innes Road at Mer Bleeu Road
Ottawa, Ontario

DATUM Geodetic, as provided by Stantec Consulting Ltd.

REMARKS

BORINGS BY Backhoe **DATE** 12 APR 06

FILE NO. PG0811

HOLE NO. TP31

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	88.63		
TOPSOIL 0.30	■								
Stiff, grey-brown **SILTY CLAY** 2.50	▨					1	87.63		
						2	86.63		
End of Test Pit TP terminated on bedrock surface @ 2.50m depth (TP dry upon completion)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

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)

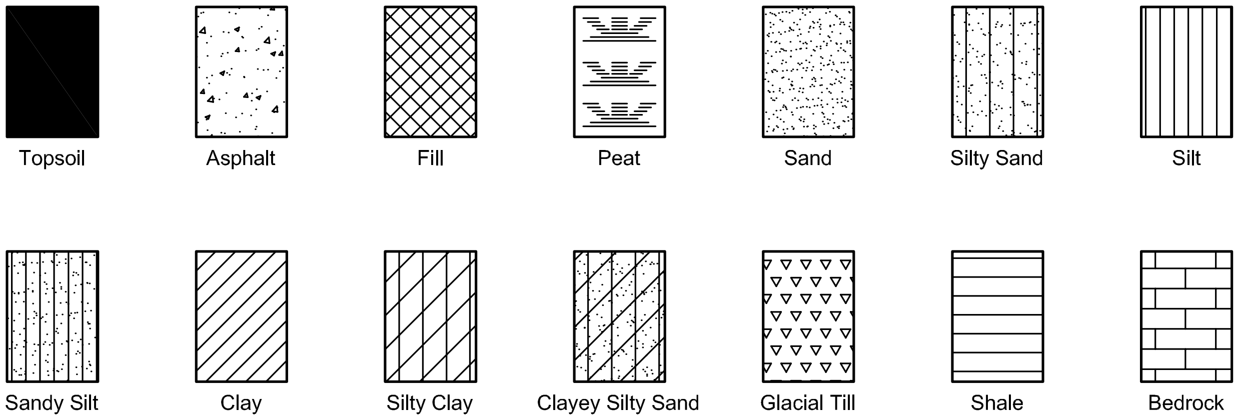
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

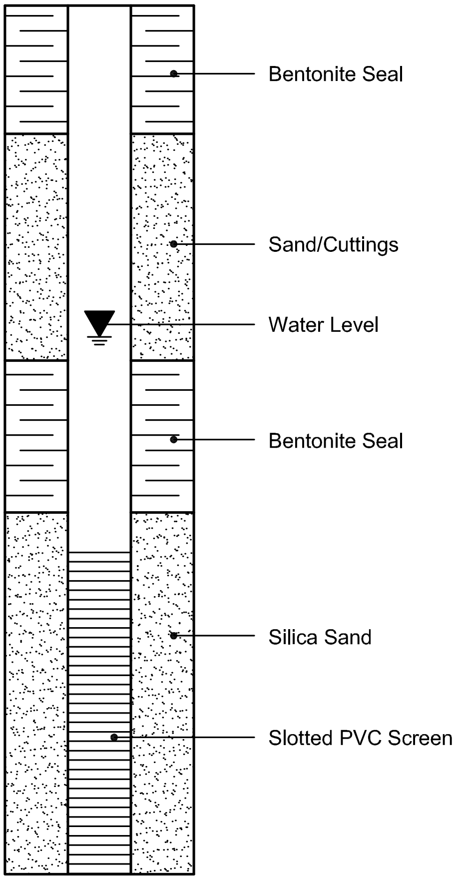
SYMBOLS AND TERMS (continued)

STRATA PLOT

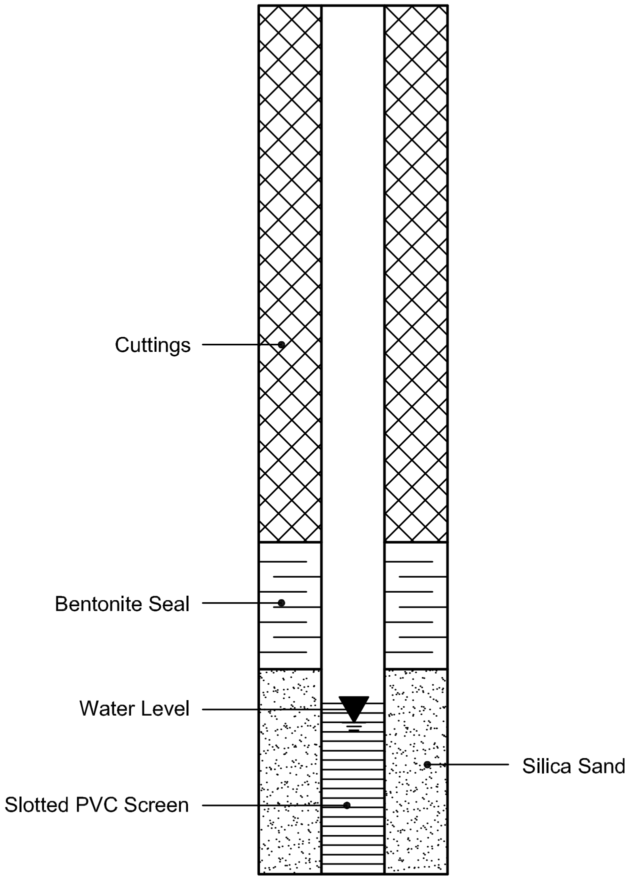


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Client ID:		BH3-22, SS5 (10'-12')	-	-	-		
Sample Date:		07-Jul-22 13:00	-	-	-	-	-
Sample ID:		2228586-01	-	-	-		
Matrix:		Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	87.9	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.46	-	-	-	-	-
Resistivity	0.1 Ohm.m	43.6	-	-	-	-	-
Anions							
Chloride	5 ug/g	929	-	-	-	-	-
Sulphate	5 ug/g	46	-	-	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURES 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG6282-1 – TEST HOLE LOCATION PLAN

DRAWING PG6282-2 – BEDROCK CONTOUR PLAN

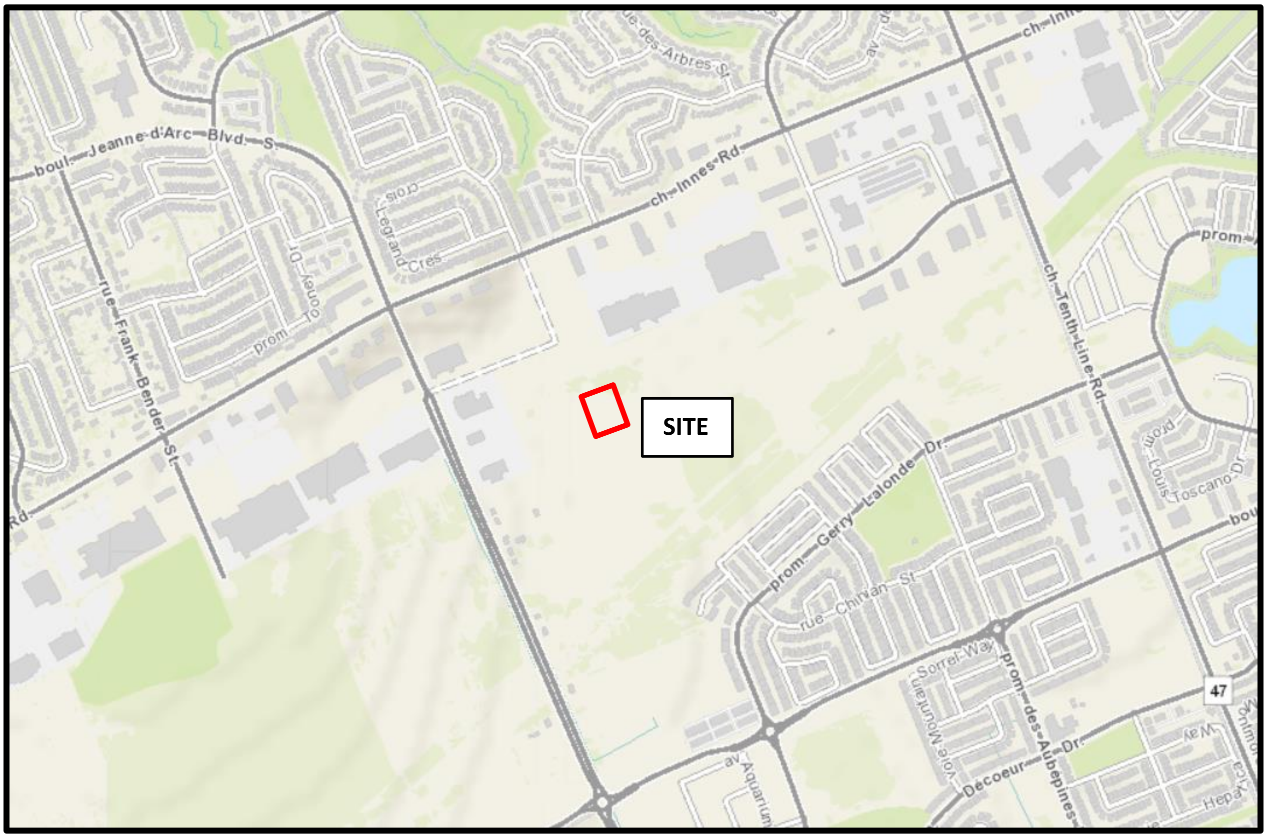


FIGURE 1

KEY PLAN

PATERSON GROUP

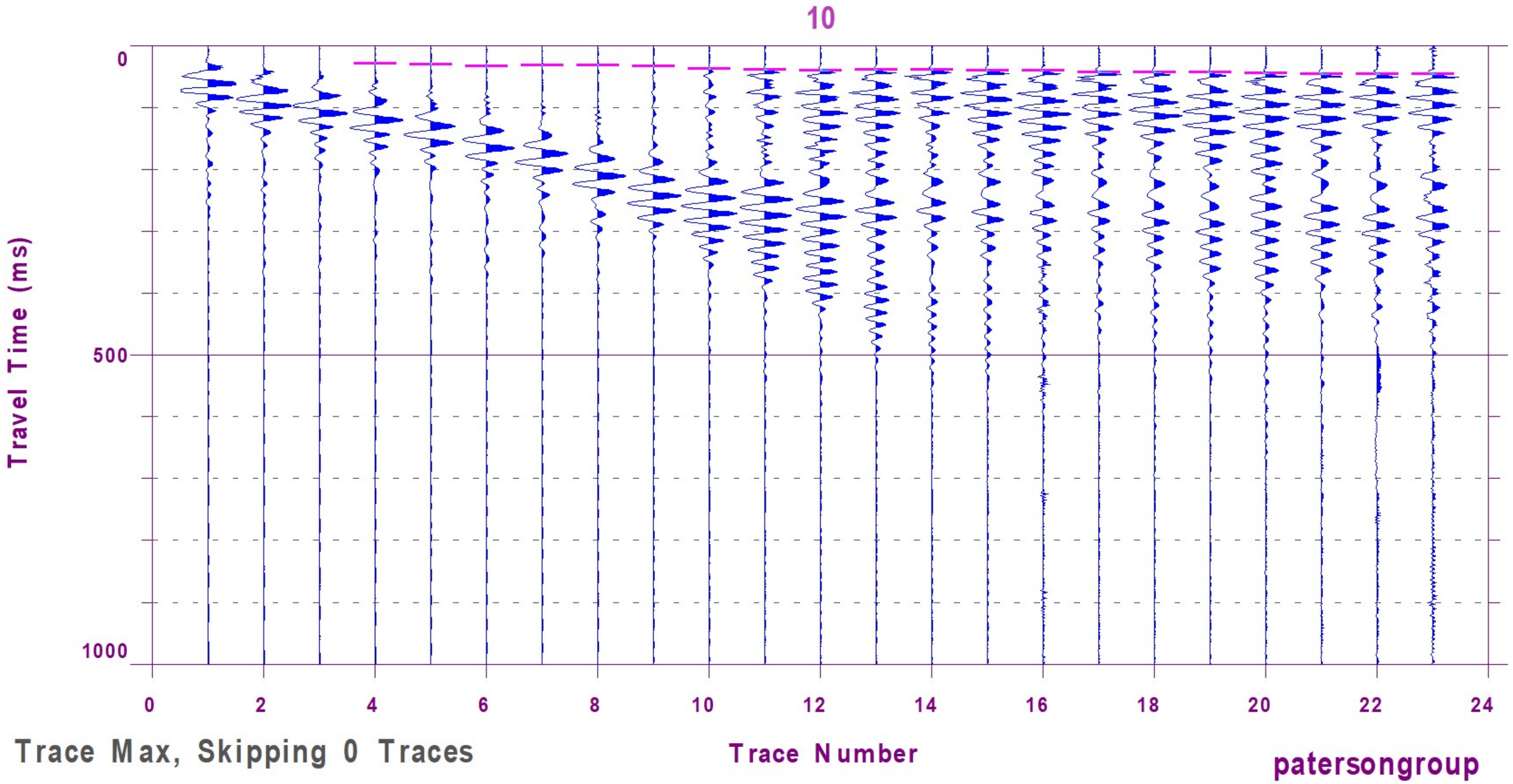


Figure 2 – Shear Wave Velocity Profile at Shot Location -2 m

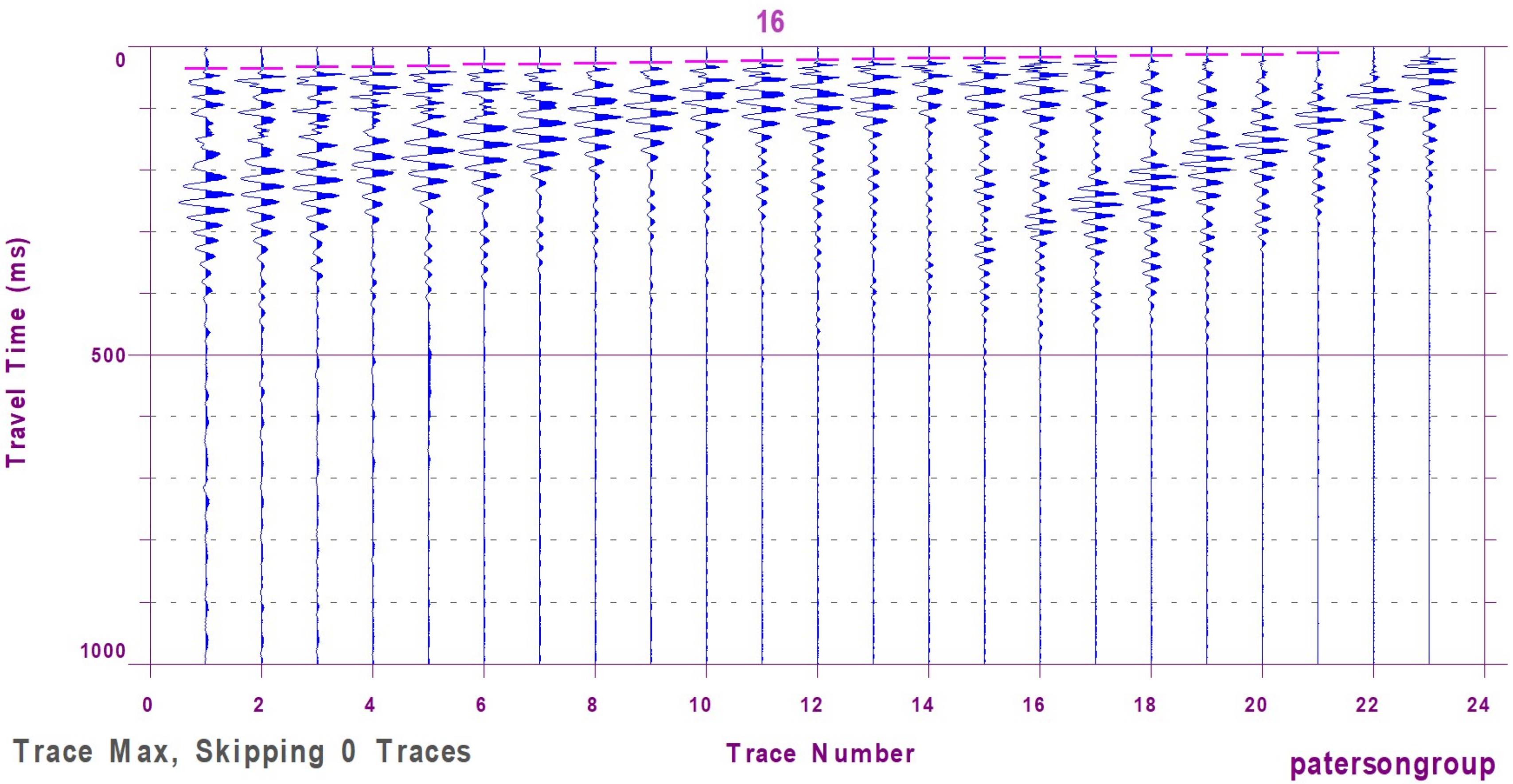
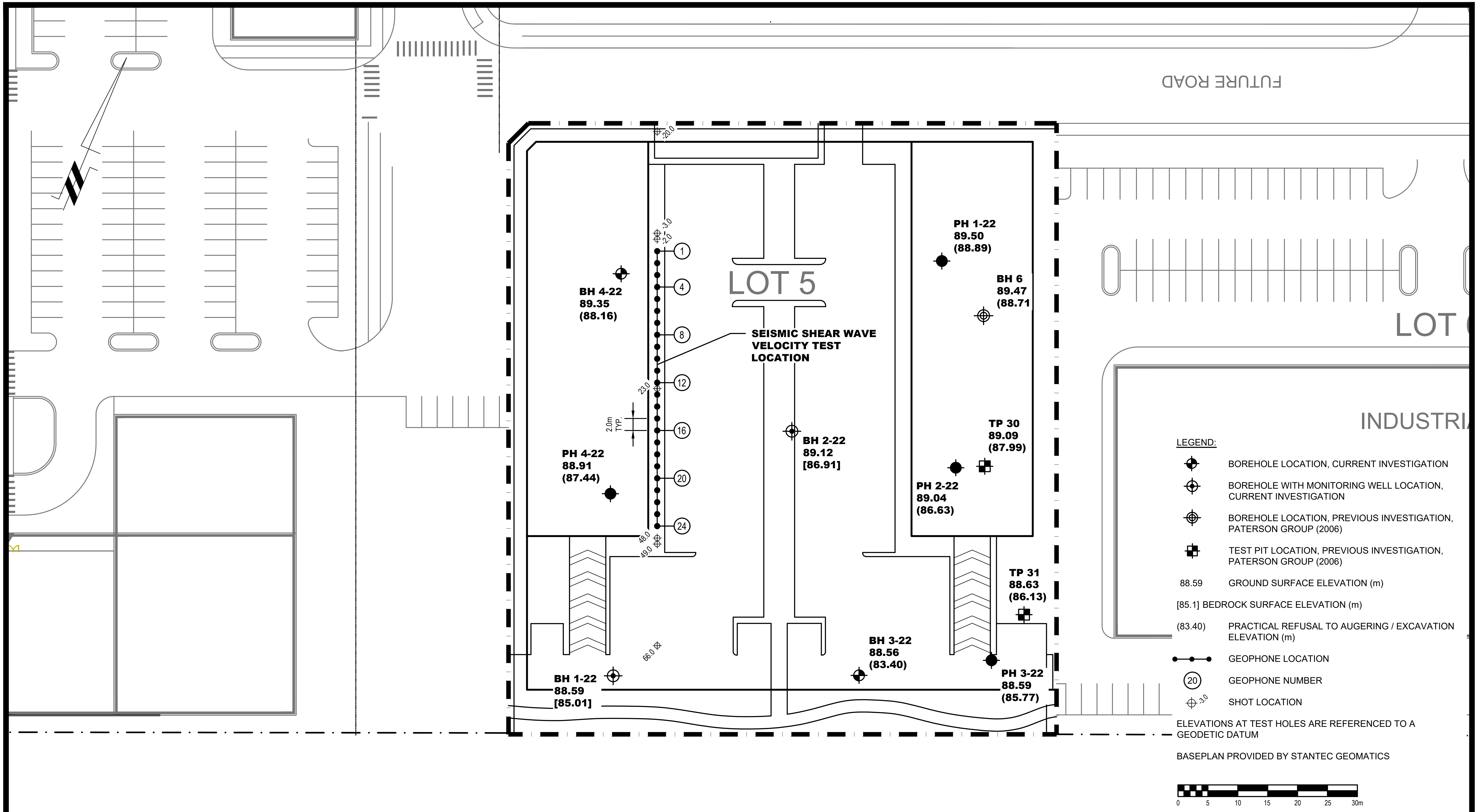


Figure 3 – Shear Wave Velocity Profile at Shot Location 48 m



PATERSON GROUP
SOLUTION ORIENTED ENGINEERING

NO.	REVISIONS	DATE	INITIAL
0			

IRONCLAD DEVELOPMENTS INC.

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDINGS - LOT 5 - 3900 INNES ROAD
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:600
Date: 07/2022
Drawn by: MPG
Report No.: PG6282-1
Checked by: DP
Dwg. No.: PG6282-1
Approved by: DP
Revision No.:

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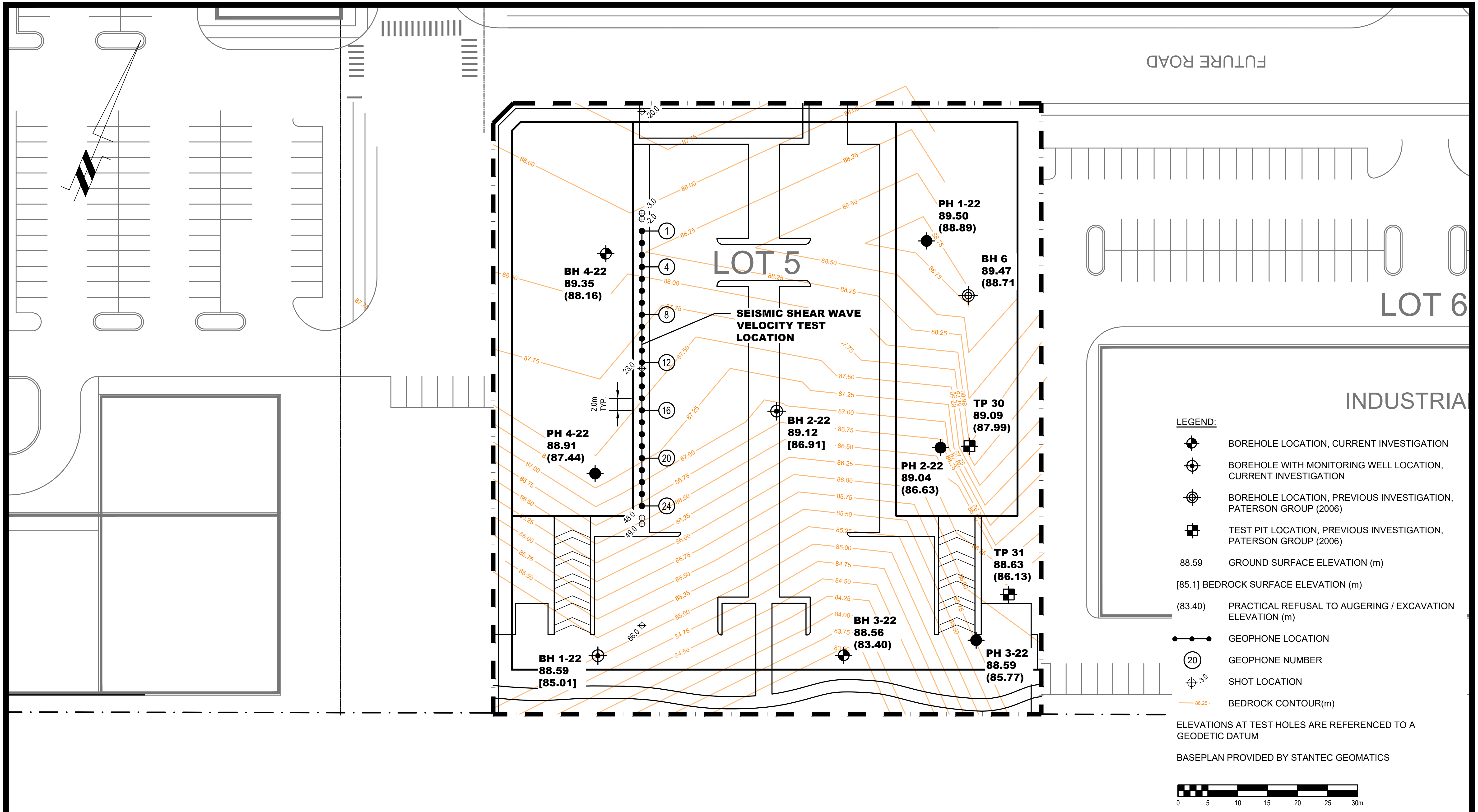

LEGEND:

◆	BOREHOLE LOCATION, CURRENT INVESTIGATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION, CURRENT INVESTIGATION
⊕	BOREHOLE LOCATION, PREVIOUS INVESTIGATION, PATERSON GROUP (2006)
▣	TEST PIT LOCATION, PREVIOUS INVESTIGATION, PATERSON GROUP (2006)
88.59	GROUND SURFACE ELEVATION (m)
[85.1]	BEDROCK SURFACE ELEVATION (m)
(83.40)	PRACTICAL REFUSAL TO AUGERING / EXCAVATION ELEVATION (m)
●—●—●	GEOPHONE LOCATION
(20)	GEOPHONE NUMBER
⊕ 3.0	SHOT LOCATION
—86.25—	BEDROCK CONTOUR(m)

ELEVATIONS AT TEST HOLES ARE REFERENCED TO A GEODETIC DATUM

BASEPLAN PROVIDED BY STANTEC GEOMATICS

PATERSON GROUP
SOLUTION ORIENTED ENGINEERING

NO.	REVISIONS	DATE	INITIAL
0			

IRONCLAD DEVELOPMENTS INC.

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDINGS - LOT 5 - 3900 INNES ROAD
OTTAWA, ONTARIO

Title:

BEDROCK CONTOUR PLAN

Scale: 1:600
Date: 07/2022
Drawn by: MPG
Report No.: PG6282-1
Checked by: DP
Dwg. No.: PG6282-2
Approved by: DP
Revision No.:

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