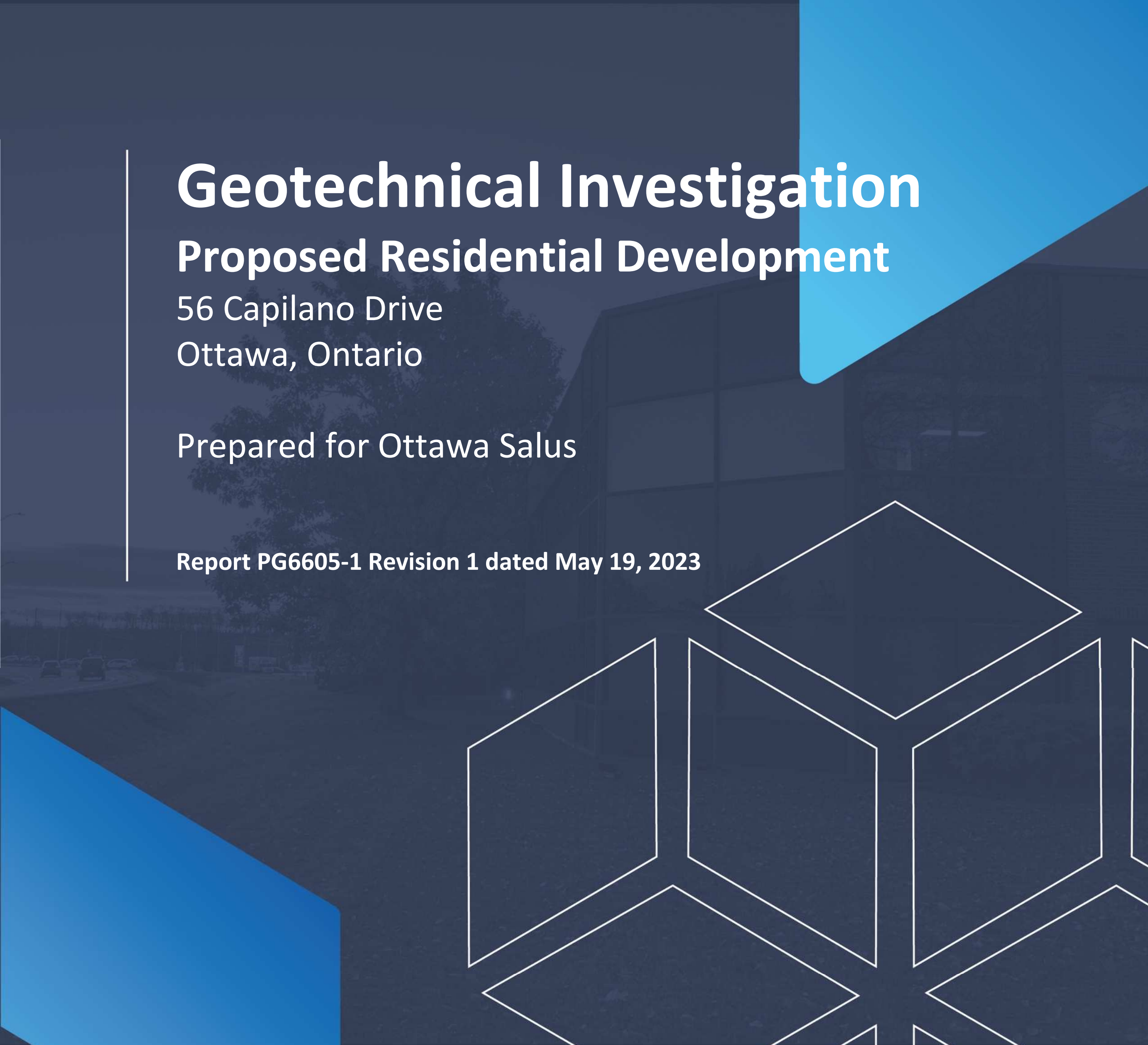

Geotechnical Investigation

Proposed Residential Development

56 Capilano Drive
Ottawa, Ontario

Prepared for Ottawa Salus

Report PG6605-1 Revision 1 dated May 19, 2023

Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Ottawa Salus to prepare a Geotechnical Investigation Report for the proposed residential development to be located at 56 Capilano Drive in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report for the general site location).

The objectives of the Geotechnical Investigation Report are to:

☐ Determine the subsoil and groundwater conditions at this site by means of existing test holes, and to

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

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

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a multi-storey building and townhouse block, each with one basement level. Associated asphalt-paved access lanes and parking areas with landscaped margins are proposed surrounding the buildings. It is expected that the proposed buildings will be municipally serviced.

3.0 Available Geotechnical Information

3.1 Field Investigation

Field Program

Previous geotechnical investigations were conducted by Paterson within the subject site, and in the vicinity of the subject site, in September 2011 (TP 1 through TP 3) and December 2013 (TP 1-13, TP 3-13, TP 5-13, and TP 6-13). Previous investigations were also completed at the subject site by others in May 2022 (TP 22-1 through TP 22-6).

All fieldwork by Paterson was conducted under the full-time supervision of our personnel under the direction of a senior engineer from the geotechnical division. The test pit procedure consisted of excavating to the required depths at the selected locations and regularly sampling the overburden. The test hole locations are shown on Drawing PG6605-1 – Test Hole Location Plan included in Appendix 2.

Reference should be made to the Record of Test Pit logs by others and the Soil Profile and Test Data sheets prepared by Paterson, which are presented in Appendix 1, for specific details of the soil profiles encountered at the test hole locations.

3.2 Field Survey

The ground surface elevations at the test hole locations were surveyed by Paterson using a temporary benchmark (TBM), consisting of the finished floor of the adjacent building, which is understood to be referenced to a geodetic elevation of 95.91 m. The locations of the test holes are presented on Drawing PG6605-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

At the times of the previous investigations, soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

One (1) soil sample was collected at an approximate depth of 1 m within the central portion of the site on May 15, 2023 using a hand auger. This sample was subsequently 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 chlorine, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site currently consists of a vacant lot with a grass surface. The site is bordered by Capilano Drive to the north, a commercial property to the west, an indoor curling rink to the south, and residential properties to the east. The ground surface across the site is generally flat and at grade with the adjacent roadway at approximate geodetic elevation 95 m.

Based on our review of available aerial photos, a former indoor rink building was located at the subject site as recently as 2015, and was demolished by 2017.

4.2 Subsurface Profile

Overburden

Generally, the subsoil conditions at the test hole locations consist of a thin layer of topsoil underlain by fill which extends to approximate depths of 1 to 1.5 m below the existing ground surface. The fill was generally observed to consist of silty sand to sandy clay with gravel and cobbles.

At test pits TP 1, TP 2 and TP 3, an approximate 0.1 to 0.3 m thick layer of stiff, grey silty clay was observed underlying the fill.

Silty sand and/or glacial till were encountered underlying the fill and/or silty clay. The glacial till, where encountered, was observed to consist of a compact to dense, grey silty clay with varying amounts of gravel, cobbles, and boulders.

Bedrock

Practical refusal to excavation on the bedrock surface was encountered at approximate depths of 2 to 2.4 m below the existing ground surface, within the subject site.

Based on the available geological mapping, the bedrock is part of the Gull River Formation and consists of interbedded limestone and dolomite.

Reference should be made to the Record of Test Pit logs by others and the Soil Profile and Test Data sheets prepared by Paterson in Appendix 1 for the details of the subsurface profile encountered at each test hole location.

4.3 Groundwater

Groundwater infiltration levels were measured in the open test holes upon the completion of excavation. The test pits were observed to be dry upon the completion of excavation. Therefore, it is expected that the long-term groundwater level is located within the bedrock.

However, it should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. The proposed buildings are recommended to be founded on conventional spread footings placed on clean, surface sounded bedrock.

If the bedrock is encountered below the underside of footing elevation, lean concrete should be used to raise grades from the clean, surface sounded bedrock to the founding elevation.

Foundations from the demolished building which formerly occupied the site may be present below the ground surface. It is recommended that an allowance be provided in the project tender for the removal of buried foundations. The outline of the former building is shown on Drawing PG6605-1 – Test Hole Location Plan included in Appendix 2.

Bedrock removal will be required to complete the basement levels. The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing 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 footprints of the proposed buildings. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming or 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.

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

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the nearby buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz).

It should be noted that these guidelines are for today’s construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Engineered fill placed for grading beneath the proposed buildings, where required, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings and paved areas should be compacted to at least 98% of the material’s standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least 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 use as backfill against foundation walls unless used in conjunction with a composite drainage blanket.

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.

Lean Concrete Filled Trenches

Where the proposed footings are to be founded on clean, surface sounded bedrock which is located below the underside of footing elevation, zero-entry vertical trenches should be excavated to the clean, surface sounded bedrock, and backfilled with lean concrete to the founding elevation (minimum **17 MPa** 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. 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. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on clean, surface sounded bedrock can be designed using a bearing resistance value at ULS of **1,500 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

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 surface sounded bedrock and designed using the above-mentioned bearing resistance values will be subjected to negligible post-construction total and differential settlements.

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 a sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium, will require a lateral support zone of 1H:1V (or shallower).

Permissible Grade Raise

Due to the presence of the silty clay deposit which was encountered at certain test pit locations, a permissible grade raise restriction of **3.0 m** is recommended for grading at the subject site.

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 in the previous investigation 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 shear wave testing location is presented in Drawing PG6605-1 - Test Hole Location Plan attached to the present report. Paterson field personnel placed 24 horizontal geophones in a straight line in roughly a north-south orientation. The 4.5 Hz. Horizontal geophones were mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 3 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 five (5) to ten (10) times at each shot location to improve signal to noise ratio.

The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at the centre of the geophone array, and 3 m and 4.5 m away from the first and last geophone.

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, Vs_{30}, of the upper 30 m 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.

Based on our analysis, the bedrock seismic shear wave velocity was calculated to be 2,300 m/s.

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

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

$$V_{s30=} \frac{30\,m}{\left(\frac{30\,m}{2{,}300\,m/s}\right)}$$

$$V_{s30=}\ 2{,}300\ m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for the proposed building is 2,300 m/s when considering footings bearing directly on the clean, surface sounded bedrock. Therefore, a **Site Class A** is applicable for 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 Floor Slab

With the removal of all topsoil and deleterious fill, such as those containing organic materials, within the footprint of the proposed buildings, the native soil surface or bedrock medium will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction.

It is recommended that the upper 200 mm of sub-slab fill below the basement slabs consist of 19 mm clear crushed stone.

In consideration of the anticipated groundwater conditions, an underslab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear crushed stone under basement slabs. 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 buildings. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 20 kN/m^3 (effective unit weight 13 kN/m^3).

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/m3)
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:

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The 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 = 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 areas and access lanes are proposed as part of the development at this site. The recommended pavement structures are shown in Tables 1 and 2 below.

Table 1 - 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 2 - Recommended Pavement Structure – Access Lanes

Thickness (mm)	Material Description
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
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 99% of the material's SPMDD using suitable vibratory equipment.

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 structures. The systems should consist of a 100 to 150 mm diameter, geotextile wrapped, 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 each structure. The pipes should have a positive outlet, such as a gravity connection to the storm sewer.

A composite foundation drainage board, such as Miradrain G100N or Delta Drain 6000, should be installed on all exterior foundation walls, from 300 mm below finished grade down to the underside of footing level.

Underslab Drainage System

An underslab drainage system is recommended to control water infiltration below the basement slabs. For design purposes, it is recommended that 150 mm perforated pipes be placed at approximate 6 m centres underlying the basement slabs.

Foundation Backfill

Where hard surfaces, such as walkways, are located directly adjacent to the building, the foundation walls should be backfilled using clean sand or OPSS Granular B Type I granular material should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

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

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

However, the footings are generally not expected to require protection against frost action due to the founding depth.

6.3 Excavation Side Slopes and Temporary Shoring

The side slopes of excavations in the 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. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

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

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

Temporary Shoring

Due to the expected depth of excavation to accommodate the basement level and the proximity of the proposed development to surrounding boundaries, temporary shoring may be required in certain areas. The design and approval of the temporary shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is 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 any 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.

The temporary shoring system, where required, may generally consist of a soldier pile and lagging system. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 3 – Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m^3	21
Effective Unit Weight (γ'), kN/m^3	13

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described above.

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.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on a soil subgrade. The bedding thickness should be increased to 300 mm when placed over a bedrock subgrade. The bedding should extend to the spring line of the pipe. 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 98% of the SPMDD.

Generally, it should be possible to re-use the excavated, on-site soils above the cover material if the excavation and filling operations are carried out in dry weather conditions.

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) 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 95% 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

It is anticipated that groundwater infiltration into the excavations should be relatively low 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.

Groundwater Control for Building Construction

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

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

If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of PTTW application.

Impacts on Neighbouring Properties

Given the shallow bedrock present at and in the vicinity of the subject site, the neighbouring structures are expected to be founded on bedrock. Therefore, no issues are expected with respect to groundwater lowering that would cause damage to adjacent structures surrounding the proposed development.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

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

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost into 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 (GU – General Use 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 slightly to moderately 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.

☐ 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 soils generated by construction activities that will be transported on-site or off-site 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 our present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

The 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 Ottawa Salus, 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.

Kevin A. Pickard, EIT

LICENSED PROFESSIONAL ENGINEER
May 19, 2023
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Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

TEST PIT LOGS BY OTHERS

ANALYTICAL TEST RESULTS

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Curling Club Facility - 50 Capilano Drive
Ottawa, Ontario

DATUM TBM - Finished floor of existing building on threshold of door. Geodetic elevation = 95.91m, as per plan provided by D.B. Grey Engineering Inc.

REMARKS

BORINGS BY Backhoe

DATE December 18, 2013

FILE NO. PG3145

HOLE NO. TP 1-13

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			20 40 60 80	
GROUND SURFACE						0	95.84		
FILL: Gravel with roots		G	1						
0.38									
FILL: Brown silty sand, some gravel, trace cobbles		G	2			1	94.84		
1.22									
Brown **SILTY SAND**									
1.52									
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders, trace clay		G	3			2	93.84		
2.46									
End of Test Pit									
TP terminated on inferred bedrock surface at 2.46m depth									
(TP dry upon completion)									

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
Prop. Curling Club Facility - 50 Capilano Drive
Ottawa, Ontario

DATUM TBM - Finished floor of existing building on threshold of door. Geodetic elevation = 95.91m, as per plan provided by D.B. Grey Engineering Inc.

REMARKS

BORINGS BY Backhoe

DATE December 18, 2013

FILE NO. PG3145

HOLE NO. TP 3-13

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	95.92		
FILL: Brown silty sand with gravel, cobbles, trace boulders		G	1						
						1	94.92		
1.55									
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders		G	2			2	93.92		
2.62									
End of Test Pit TP terminated on inferred bedrock surface at 2.62m depth (TP dry upon completion)									

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
Prop. Curling Club Facility - 50 Capilano Drive
Ottawa, Ontario

DATUM TBM - Finished floor of existing building on threshold of door. Geodetic elevation = 95.91m, as per plan provided by D.B. Grey Engineering Inc.

REMARKS

BORINGS BY Backhoe

DATE December 18, 2013

FILE NO. PG3145

HOLE NO. TP 5-13

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	95.80		
FILL: Brown silty sand with gravel, cobbles, trace boulders		G G	1 2			1	94.80		
1.09									
Dense, brown **SILT**		G G	3 4						
1.68									
GLACIAL TILL: Brown silty sand with gravel, cobbles and boulders, trace clay		G	5			2	93.80		
2.82									
End of Test Pit TP terminated on inferred bedrock surface at 2.82m depth (TP dry upon completion)									

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
Prop. Curling Club Facility - 50 Capilano Drive
Ottawa, Ontario

DATUM TBM - Finished floor of existing building on threshold of door. Geodetic elevation = 95.91m, as per plan provided by D.B. Grey Engineering Inc.

REMARKS

BORINGS BY Backhoe

DATE December 18, 2013

FILE NO. PG3145

HOLE NO. TP 6-13

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	95.58		
FILL: Brown silty sand with gravel, cobbles, boulders, trace clay		G	1			1	94.58		
1.09									
Brown **SILTY SAND**		G	2						
1.40									
End of Test Pit (TP dry upon completion)									

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 Curling Club Facility - 50 Capilano Drive
Ottawa, Ontario

DATUM Ground surface elevations at test hole locations are inferred based on topographic mapping provided by D.B. Grey Engineering Inc.

REMARKS

BORINGS BY Backhoe

DATE September 8, 2011

FILE NO. PG2655

HOLE NO. TP 1

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	95.70		
TOPSOIL 0.13									
FILL: Silty sand with gravel, cobbles and boulders 0.94									
Compact **SILTY SAND** 1.45						1	94.70		
Stiff, grey **SILTY CLAY** 1.65		G	1						
GLACIAL TILL: Compact, silty clay with gravel, cobbles and boulders 2.54		G	2			2	93.70		
End of Test Pit TP terminated on bedrock surface @ 2.54m depth (TP dry upon completion)									

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 Curling Club Facility - 50 Capilano Drive
Ottawa, Ontario

DATUM Ground surface elevations at test hole locations are inferred based on topographic mapping provided by D.B. Grey Engineering Inc.

REMARKS

BORINGS BY Backhoe

DATE September 8, 2011

FILE NO. PG2655

HOLE NO. TP 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	95.80
TOPSOIL 0.10							
FILL: Silty sand with gravel, cobbles and boulders 1.07						1	94.80
Stiff, grey **SILTY CLAY** 1.42		G	1				
GLACIAL TILL: Compact, silty clay with sand gravel, cobbles and boulders		G	2			2	93.80
2.69		G	3				
End of Test Pit TP terminated on bedrock surface @ 2.69m depth (TP dry upon completion)							

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

○ **Water Content %**

20 40 60 80

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

Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Curling Club Facility - 50 Capilano Drive
Ottawa, Ontario

DATUM Ground surface elevations at test hole locations are inferred based on topographic mapping provided by D.B. Grey Engineering Inc.

REMARKS

BORINGS BY Backhoe

DATE September 8, 2011

FILE NO. PG2655

HOLE NO. TP 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	95.70
TOPSOIL 0.10							
FILL: Brown silty sand with gravel and cobbles						1	94.70
1.55							
Stiff, grey **SILTY CLAY** 1.63		G	1				
GLACIAL TILL: Compact to dense silty clay with sand, gravel, cobbles and boulders						2	93.70
2.39		G	2				
End of Test Pit TP terminated on bedrock surface @ 2.39m depth (TP dry upon completion)							

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

○ **Water Content %**

20 40 60 80

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

Piezometer Construction

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity: $S_t < 2$
Medium Sensitivity: $2 < S_t < 4$
Sensitive: $4 < S_t < 8$
Extra Sensitive: $8 < S_t < 16$
Quick Clay: $S_t > 16$

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW - Thin wall tube or Shelby tube, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

Cc - Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu - Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

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

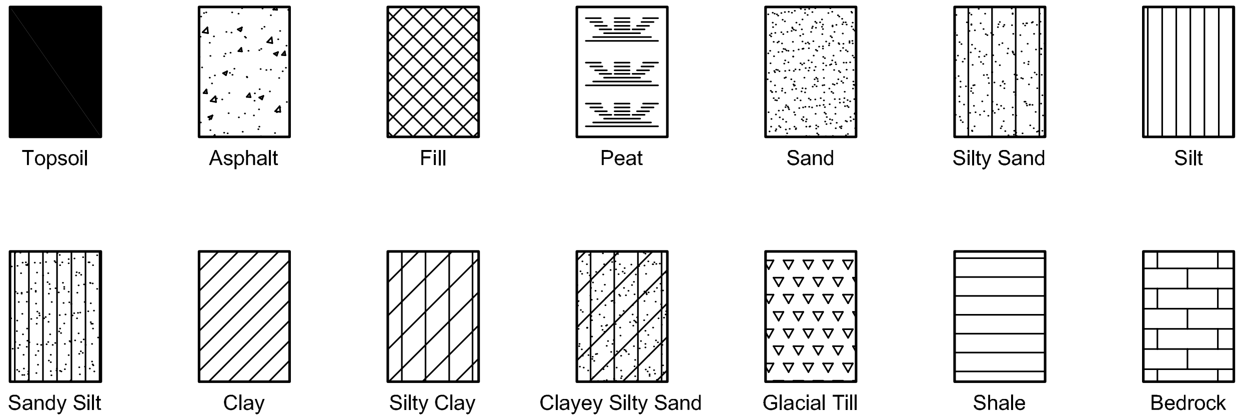
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

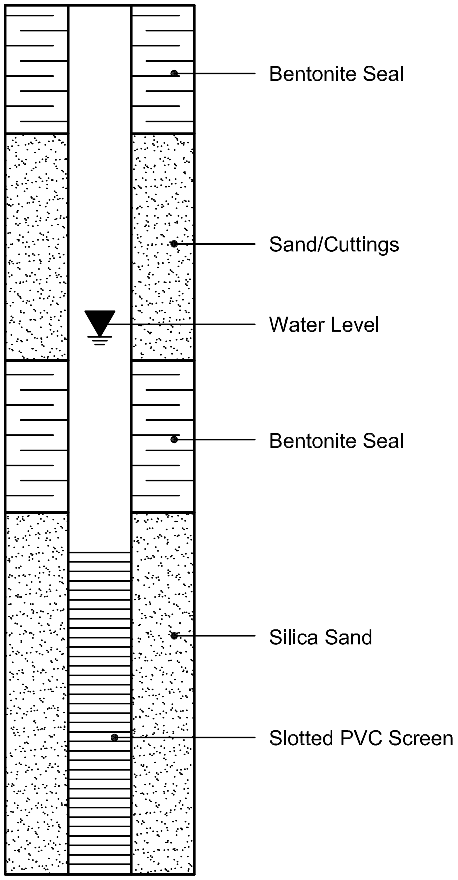
SYMBOLS AND TERMS (continued)

STRATA PLOT

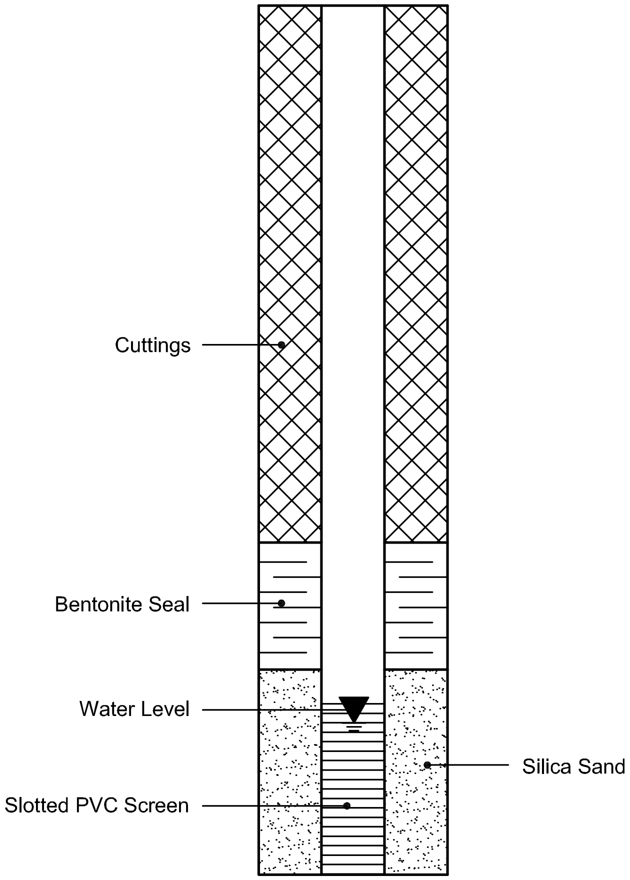


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Log of Test Pit TP22-01

Project No: OTT-22003851-A0

Project: Phase Two Environmental Site Assessment

Location: 56 Cpailano Drive, Ottawa, ON

Figure No. C-1

Page. 1 of 1

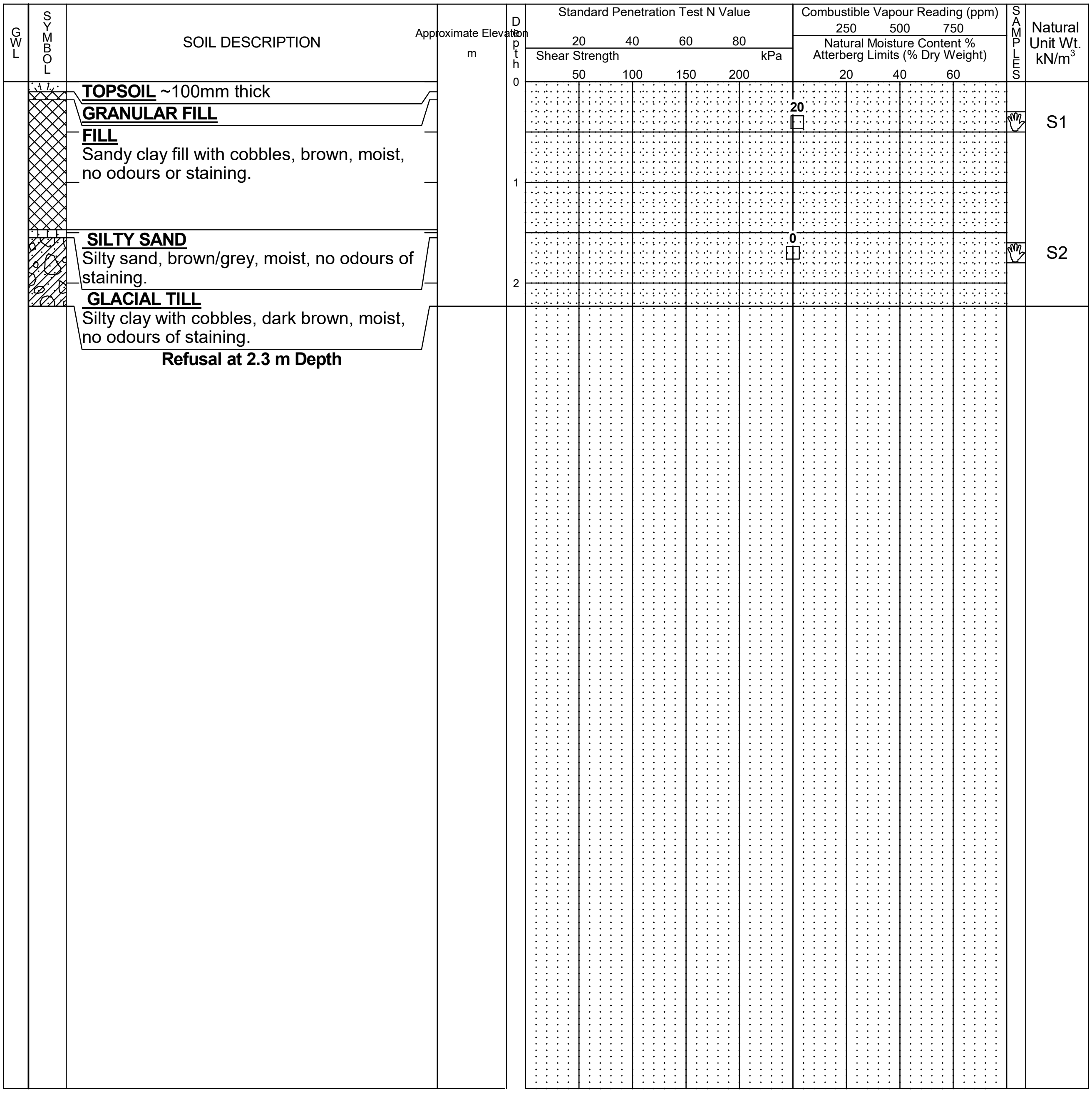
Date Drilled: 5/16/22

Drill Type:

Datum: Relative

Logged by: L.W. Checked by: M.M.

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	▥	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲



TOPSOIL ~100mm thick

GRANULAR FILL

FILL
Sandy clay fill with cobbles, brown, moist, no odours or staining.

SILTY SAND
Silty sand, brown/grey, moist, no odours of staining.

GLACIAL TILL
Silty clay with cobbles, dark brown, moist, no odours of staining.

Refusal at 2.3 m Depth

NOTES:
1. Borehole/Test Pit data requires Interpretation by exp. before use by others
2. Test pit was backfilled upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-22003851-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Test Pit TP22-02

Project No: OTT-22003851-A0

Project: Phase Two Environmental Site Assessment

Location: 56 Cpailano Drive, Ottawa, ON

Figure No. C-2

Page. 1 of 1

Date Drilled: 5/16/22

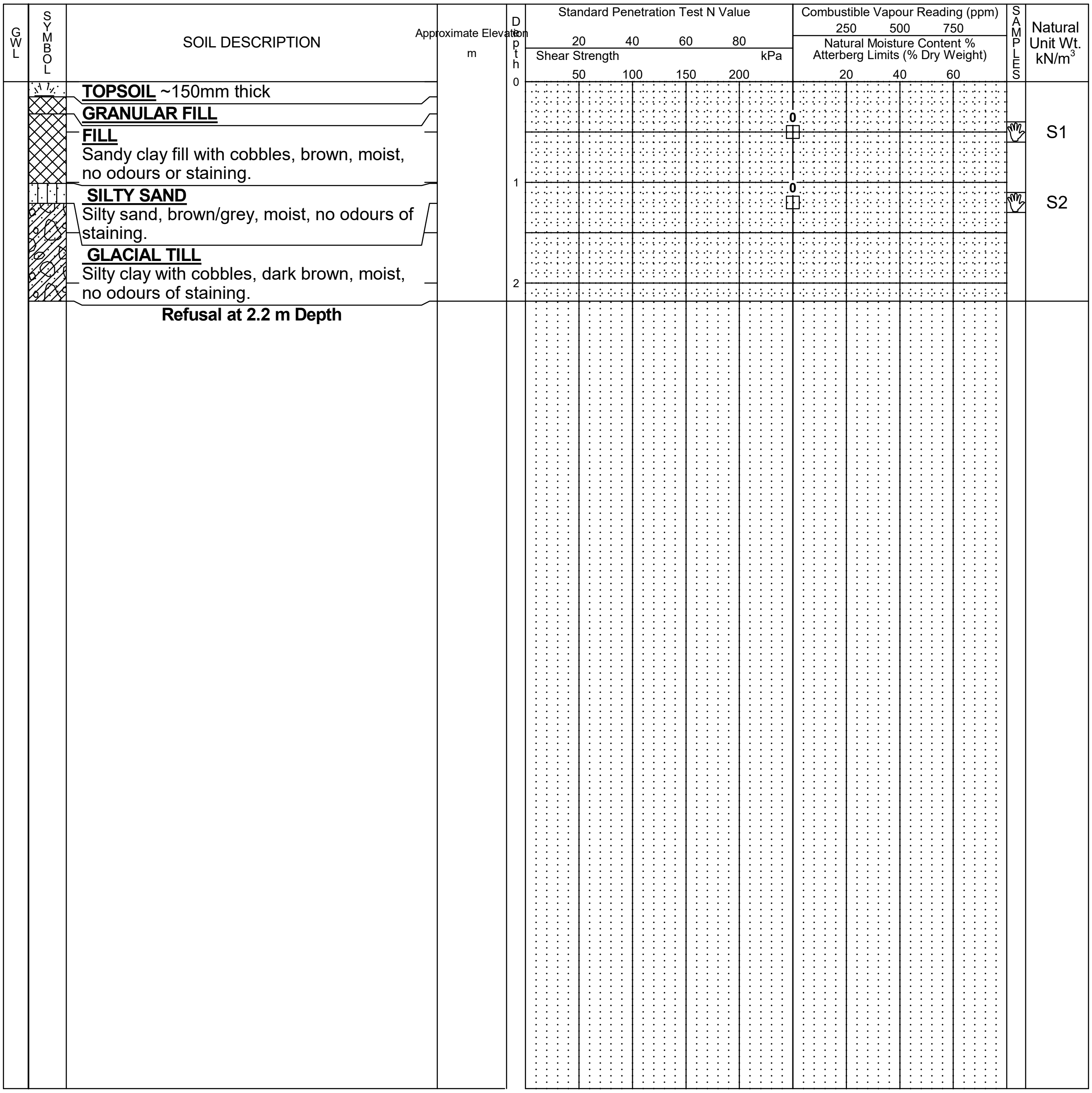
Drill Type:

Datum: Relative

Logged by: L.W. Checked by: M.M.

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

GWL	SYMBOL	SOIL DESCRIPTION	Approximate Elevation m	Depth
		TOPSOIL ~150mm thick		0
		GRANULAR FILL		
		FILL Sandy clay fill with cobbles, brown, moist, no odours or staining.		
		SILTY SAND Silty sand, brown/grey, moist, no odours of staining.		1
		GLACIAL TILL Silty clay with cobbles, dark brown, moist, no odours of staining.		2
		Refusal at 2.2 m Depth		



NOTES:
1. Borehole/Test Pit data requires Interpretation by exp. before use by others
2. Test pit was backfilled upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-22003851-A0

WATER LEVEL RECORDS		
Elapsed Time	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD			
Run No.	Depth (m)	% Rec.	RQD %

Log of Test Pit TP22-03

Project No: OTT-22003851-A0

Project: Phase Two Environmental Site Assessment

Location: 56 Cpailano Drive, Ottawa, ON

Figure No. C-3

Page. 1 of 1

Date Drilled: 5/16/22

Drill Type:

Datum: Relative

Logged by: L.W. Checked by: M.M.

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	⊢⊖
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		

GWL	SYMBOL	SOIL DESCRIPTION	Approximate Elevation m	Depth	Standard Penetration Test N Value 20 40 60 80 Shear Strength kPa 50 100 150 200	Combustible Vapour Reading (ppm) 250 500 750 Natural Moisture Content % Atterberg Limits (% Dry Weight) 20 40 60	SAMPLES	Natural Unit Wt. kN/m^3
		TOPSOIL ~180mm thick		0				
		GRANULAR FILL						
		FILL Sandy clay fill with cobbles, brown, moist, no odours or staining.		1		0		S1
		SILTY SAND Silty sand, brown/grey, moist, no odours of staining.		2		0		S2
		Refusal at 2.1 m Depth						

NOTES:
1. Borehole/Test Pit data requires Interpretation by exp. before use by others
2. Test pit was backfilled upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-22003851-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Test Pit TP22-04

Project No: OTT-22003851-A0

Project: Phase Two Environmental Site Assessment

Location: 56 Cpailano Drive, Ottawa, ON

Figure No. C-4

Page. 1 of 1

Date Drilled: 5/16/22

Drill Type:

Datum: Relative

Logged by: L.W. Checked by: M.M.

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Approximate Elevation m	Depth	Standard Penetration Test N Value 20 40 60 80 Shear Strength kPa 50 100 150 200	Combustible Vapour Reading (ppm) 250 500 750 Natural Moisture Content % Atterberg Limits (% Dry Weight) 20 40 60	SAMPLES	Natural Unit Wt. kN/m^3
		TOPSOIL ~180mm thick		0				
		FILL Sandy clay fill with cobbles, brown, moist, no odours or staining.		1		0		S1
		GLACIAL TILL Silty clay with cobbles, dark brown, moist, no odours of staining.		2		0		S2
		Refusal at 2.4 m Depth						

NOTES:
1. Borehole/Test Pit data requires Interpretation by exp. before use by others
2. Test pit was backfilled upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-22003851-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Test Pit **TP22-05**

Project No: OTT-22003851-A0

Project: Phase Two Environmental Site Assessment

Location: 56 Cpailano Drive, Ottawa, ON

Figure No. C-5

Page. 1 of 1

Date Drilled: 5/16/22

Drill Type:

Datum: Relative

Logged by: L.W. Checked by: M.M.

Split Spoon Sample	☒	Combustible Vapour Reading	□
Auger Sample	◨	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Approximate Elevation m	Depth	Standard Penetration Test N Value 20 40 60 80 Shear Strength kPa 50 100 150 200	Combustible Vapour Reading (ppm) 250 500 750 Natural Moisture Content % Atterberg Limits (% Dry Weight) 20 40 60	SAMPLES	Natural Unit Wt. kN/m^3
		TOPSOIL ~200mm thick		0				
		FILL Sandy clay fill with cobbles, brown, moist, no odours or staining, some minor debris (<1%).		1		0		S1
		SILTY SAND Silty sand, brown/grey, moist, no odours of staining.				0		S2
		Refusal at 2.0 m Depth						

NOTES:
1. Borehole/Test Pit data requires Interpretation by exp. before use by others
2. Test pit was backfilled upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-22003851-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)

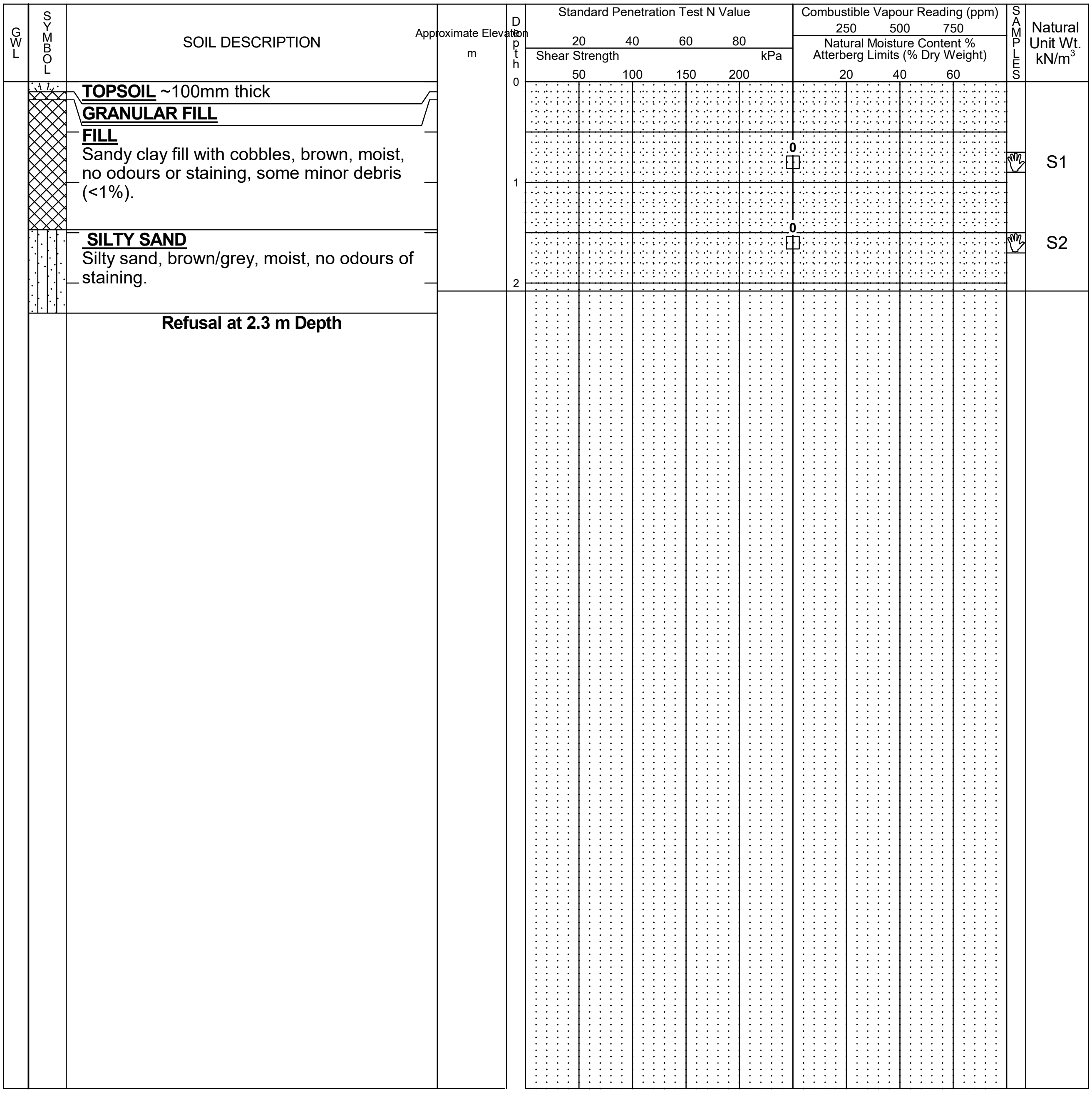
CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Test Pit TP22-06

Project No:	OTT-22003851-A0	Figure No.	C-6
Project:	Phase Two Environmental Site Assessment	Page.	1 of 1
Location:	56 Cpailano Drive, Ottawa, ON		
Date Drilled:	5/16/22		
Drill Type:			
Datum:	Relative		
Logged by:	L.W.	Checked by:	M.M.

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◫	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├──⊖
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



TOPSOIL ~100mm thick

GRANULAR FILL

FILL
Sandy clay fill with cobbles, brown, moist, no odours or staining, some minor debris (<1%).

SILTY SAND
Silty sand, brown/grey, moist, no odours of staining.

Refusal at 2.3 m Depth

NOTES:
1. Borehole/Test Pit data requires Interpretation by exp. before use by others
2. Test pit was backfilled upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. This Figure is to read with exp. Services Inc. report OTT-22003851-A0

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 57475

Report Date: 19-May-2023

Order Date: 15-May-2023

Project Description: PG6605

	Client ID:	TP22-2 @1m	-	-	-
	Sample Date:	15-May-23 00:00	-	-	-
	Sample ID:	2320069-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	86.2	-	-	-
General Inorganics					
pH	0.05 pH Units	7.41	-	-	-
Resistivity	0.1 Ohm.m	57.2	-	-	-
Anions					
Chloride	10 ug/g dry	<10	-	-	-
Sulphate	10 ug/g dry	25	-	-	-

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG6605-1 – TEST HOLE LOCATION PLAN

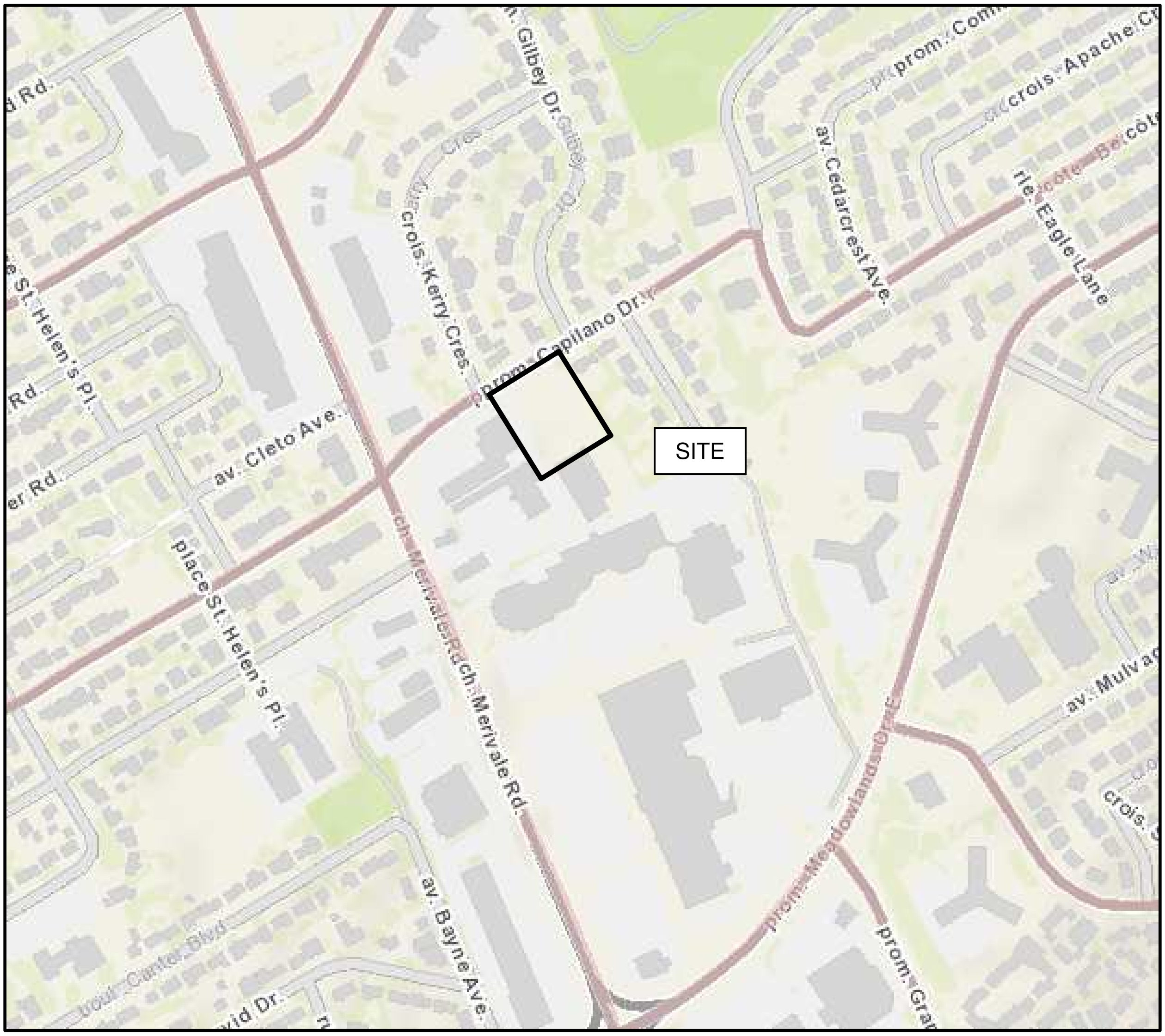


FIGURE 1

KEY PLAN

PATERSON
GROUP

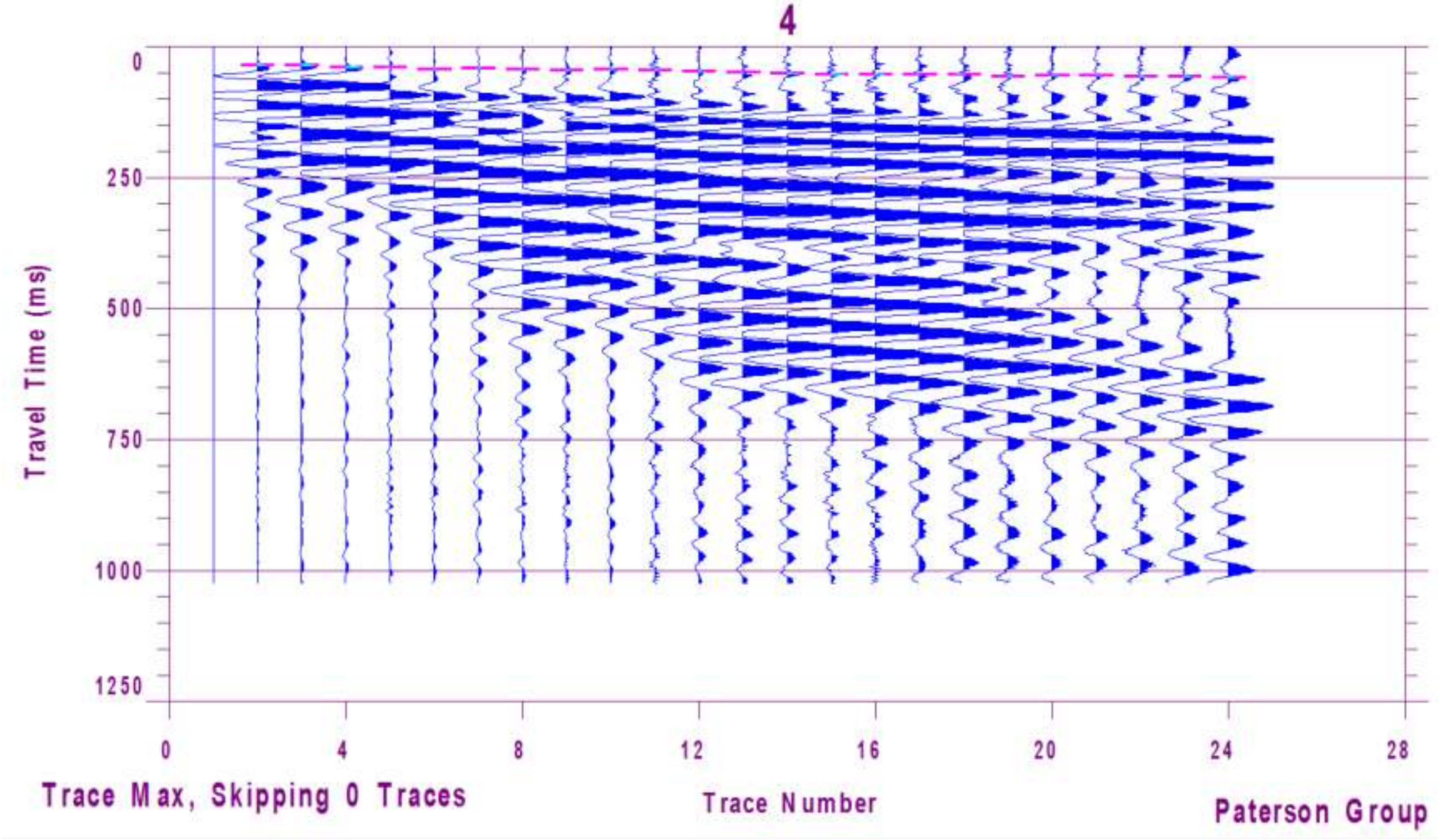


Figure 2 – Shear Wave Velocity Profile at Shot Location – 3 m

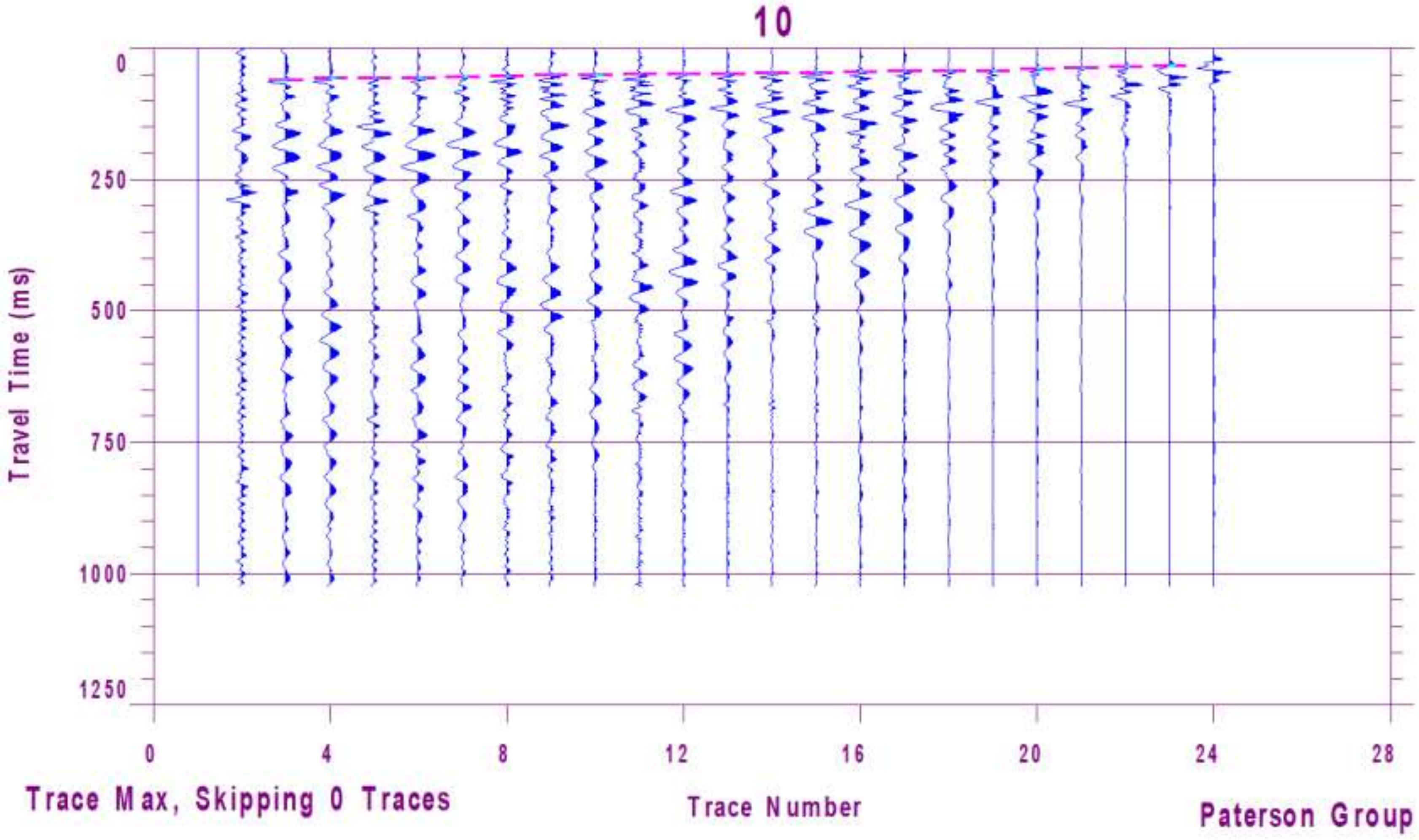
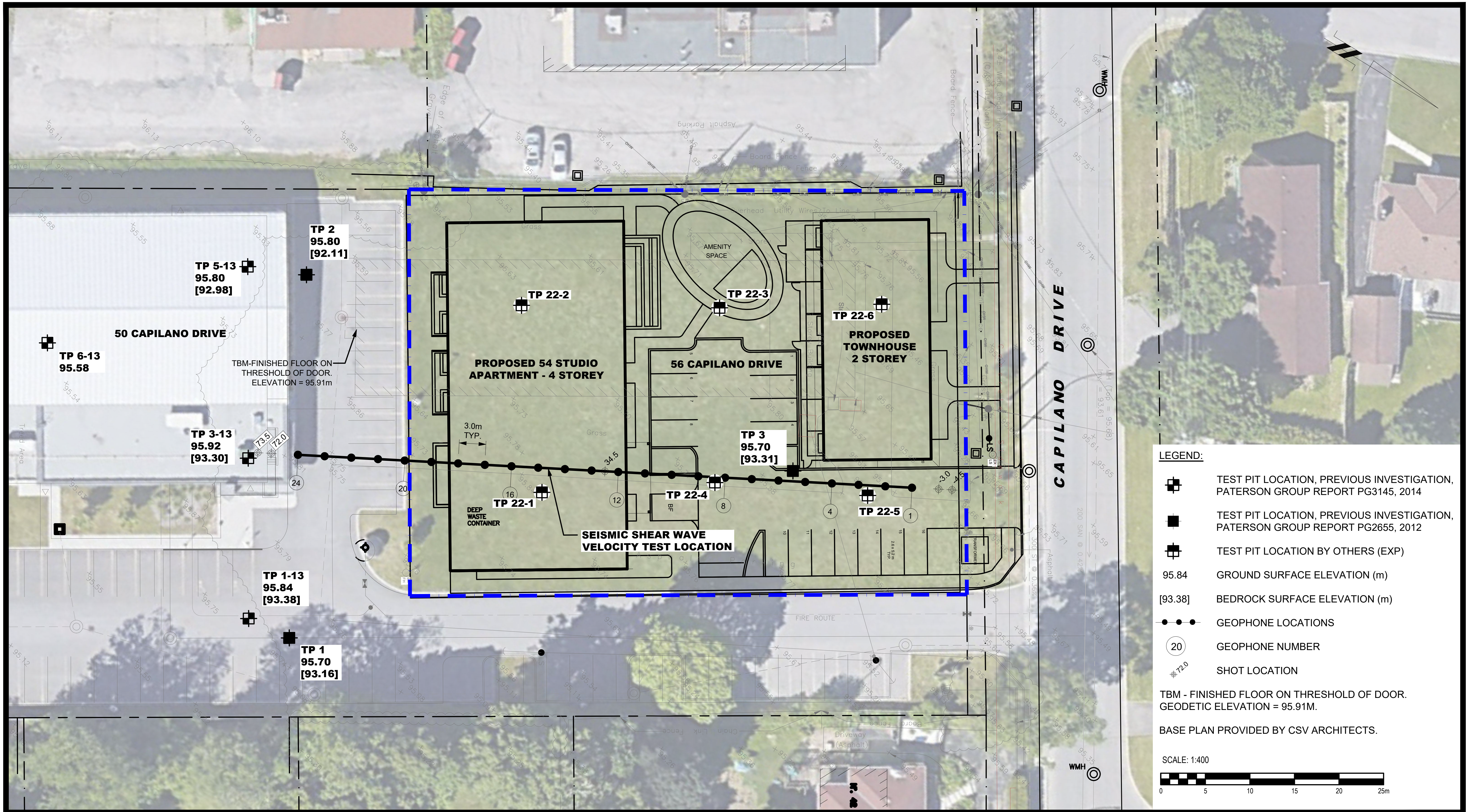


Figure 3 – Shear Wave Velocity Profile at Shot Location 73.5 m



LEGEND:

⊕	TEST PIT LOCATION, PREVIOUS INVESTIGATION, PATERSON GROUP REPORT PG3145, 2014
■	TEST PIT LOCATION, PREVIOUS INVESTIGATION, PATERSON GROUP REPORT PG2655, 2012
⊕	TEST PIT LOCATION BY OTHERS (EXP)
95.84	GROUND SURFACE ELEVATION (m)
[93.38]	BEDROCK SURFACE ELEVATION (m)
●—●—●	GEOPHONE LOCATIONS
(20)	GEOPHONE NUMBER
72.0	SHOT LOCATION

TBM - FINISHED FLOOR ON THRESHOLD OF DOOR. GEODETIC ELEVATION = 95.91M.

BASE PLAN PROVIDED BY CSV ARCHITECTS.

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

NO.	REVISIONS	DATE	INITIAL

OTTAWA SALUS
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
56 CAPILANO DRIVE
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:400
Drawn by: GK
Checked by: MA
Approved by: SD
Date: 02/2023
Report No.: PG6605-1
Dwg. No.: PG6605-1
Revision No.:

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