Geotechnical Engineering

Environmental Engineering

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Materials Testing

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Noise and Vibration Studies

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Geotechnical Investigation

Proposed Multi-Storey Building
797 Richmond Road
Ottawa, Ontario

Prepared For

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Dentech Holdings Inc. to conduct a geotechnical investigation for the proposed multi-storey building to be located at 797 Richmond Road, in the city of Ottawa, Ontario (refer to Figure 1 - Key Plan presented in Appendix 2).

The objectives of the geotechnical investigation were to:

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

- ❑ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

The following report has been prepared specifically and solely for the aforementioned project. This report contains geotechnical findings and includes recommendations pertaining to the design and construction of the proposed 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 for this current geotechnical investigation. Therefore the current report does not address environmental concerns.

2.0 Proposed Development

It is understood that the proposed project will consist of a multi-storey building with two levels of underground parking. It is expected that the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on March 3 and 4, 2021. At that time, four (4) boreholes were advanced to a maximum depth of 9.9 m below the existing ground surface. The test hole locations were distributed across the site in a manner to provide general coverage of the subject site. The locations of the test holes are shown on Drawing PG5719-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a low-clearance track mounted auger drill rig operated by a two person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Bedrock samples were recovered using a core barrel and diamond drilling techniques. The depths at which rock core samples were recovered from the boreholes are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

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

Groundwater

Monitoring wells were installed in three (3) of the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. Ground observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

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

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevations at each test hole location were surveyed by Paterson and are referenced to a geodetic datum. The approximate location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5719-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil and bedrock samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. Soil and bedrock samples will be stored for a period of one month after this report is completed, unless otherwise directed.

3.4 Analytical Testing

One 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. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. 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 is currently occupied by a one storey commercial building with an associated asphalt covered access lane and parking area. The subject site is bordered by a mid-rise building to the east, a gravel parking lot to the north, an automobile service garage to the west and Richmond Road to the south. The existing ground surface across the subject site is relatively flat and at-grade with Richmond Road. It is further understood based on available drawings that a 1200 mm watermain is located in the right-of-way approximately 3 m north of the site boundary.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the borehole locations consists of a 80 to 100 mm thick asphalt surface underlain by a fill layer extending to approximate depths of 1.5 to 2.2 m. The fill was generally observed to consist of a brown silty sand to silty clay with some crushed stone and gravel.

The fill layer was observed to be underlain by a deposit of glacial till consisting of brown to grey compact silty sand, with clay, gravel, cobbles and boulders. The glacial till deposit was encountered at approximate depths ranging from 1.5 to 2.3 m below the existing ground surface at all boreholes locations.

Practical refusal to augering was encountered in boreholes at approximate depths ranging from 5.4 and 6.4 m below the existing ground surface.

Bedrock

Bedrock was encountered in boreholes BH 2-21, BH 3-21 and BH 4-21 at depths ranging from 5.9 m to 7.3 m below existing ground surface. Bedrock was cored within these boreholes to a maximum depth of 9.9 m and was observe to consist of grey dolostone with interbedded black shale. Based on the RQD values, the bedrock cores were generally noted to be of good to excellent condition.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded dolostone and limestone from the Gull River Formation at depths ranging from 5 to 10 m.

Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

4.3 Groundwater

Groundwater level readings were measured at the monitoring wells locations on March 16, 2021. The observed groundwater levels are summarized in Table 1 below.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 2-21	64.11	4.87	59.24	March 16, 2021
BH 3-21	64.27	7.36	56.91	March 16, 2021
BH 4-21	64.07	6.10	57.97	March 16, 2021

It should be noted that groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. The long-term groundwater level can also be estimated based on the recovered soil samples' moisture levels, colouring and consistency. Based on these observations, the long-term groundwater level is anticipated at a depth of approximately 5 to 6 m below ground surface. However, groundwater levels are subject to seasonal fluctuations and 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 project. It is anticipated that the proposed multi-storey building will be founded on conventional spread footings placed on a clean, undisturbed, compact glacial till or bedrock bearing surface.

Where bedrock is not encountered at the design underside of footing elevation or the proposed building exceed the bearing resistance values provided herein for the undisturbed, compact glacial till, consideration should be taken to transferring the building loads to the bedrock surface by means of a lean concrete in-filled trench. Our foundation design recommendations are further detailed in Subsection 5.3.

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 materials, should be stripped from under any buildings and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the subgrade level during site preparation activities.

Fill Placement

Fill placed for grading beneath the structure(s) or other settlement sensitive areas should consist, unless otherwise specified, 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 the delivery to the site. The engineered fill should be placed in maximum 300 mm thick lifts and compacted to 98% of the materials Standard Proctor Maximum Dry Density (SPMDD).

Non-specified existing fill along with site-excavate soil can be placed as general landscaping fill where surface settlement is a minor concern. The backfill should be spread in thin lifts and, at minimum, compacted by the tracks of the spreading equipment to minimize voids. If the non-specified fill is to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm lifts and compacted to 95% of the material's SPMDD. Non-specified existing fill and site excavated soils are not suitable for placement as backfill against foundations walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling, controlled blasting and/or hoe ramming.

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

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm per second 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 also 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 source of detrimental vibrations on the adjoining 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). These guidelines are for current 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 preconstruction survey be completed to minimize the risks of claims during or following the construction of the proposed buildings.

Vibration and monitoring criteria for the 1200 mm watermain is discussed in Section 6.6.

5.3 Foundation Design

Bearing Resistance Values

Footings placed directly on clean, surface sounded bedrock, can be designed using a factored bearing resistance value at Ultimate Limit States (ULS) of **2,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

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.

Lean Concrete Trenches

Where bedrock is encountered below the design underside of footing elevation or should the bearing pressures from the proposed building exceed the bearing resistance values provided herein for the undisturbed, compact glacial till, the conventional spread footings are recommended to be supported on lean concrete trenches which extend to the bedrock.

In this case, as the bedrock is anticipated to be encountered below the underside of footing elevation, zero-entry vertical trenches would be excavated to the clean, surface-sounded bedrock, and backfilled with lean concrete to the founding elevation (minimum **15 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 300 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.

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, a test pit should be undertaken to assess the water infiltration issues and stability of the excavation sidewalls extending to the bedrock surface.

Footings placed on lean concrete filled trenches extending to the bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,000 kPa.**

Shallow Footings on Glacial Till Bearing Surface

Footings placed on an undisturbed, compact glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious material, such as loose, frozen or disturbed soil, whether in situ or not, have been removed in dry conditions, prior to the placement of concrete for footings.

Soil/Bedrock Transition Areas

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long term total and differential settlements. Also, at the soil/bedrock and bedrock/soil transitions, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material. The width of the subexcavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

Settlement

Footings placed on a soil bearing surface and designed using the bearing resistance values at SLS given for the soil bearing surface will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Footings bearing on directly on a sound bedrock or lean concrete trenches placed directly on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein 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 compact to dense silty sand and glacial till bearing surface above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher bearing capacity as the bearing medium soil.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge 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 bearing medium will require a lateral support zone of 1H:1V (or flatter).

5.4 Design for Earthquakes

For design purposes, the site class for seismic site response can be taken as **Class C** for the foundations considered at this site. A higher site class, such as Class A or B, is possible for footings placed within 3 m of the bedrock surface. However, the higher seismic site class would need to be confirmed by site-specific shear wave velocity testing. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill from within the footprint of the proposed building, the native soil or bedrock surface will be considered an acceptable subgrade on which to commence backfilling for 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 clear crushed stone.

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

In consideration of the groundwater conditions encountered at the time of the fieldwork, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a sump pit, should be provided in the clear stone under the lower parking level. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed (discussed further in Subsection 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 subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. The applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

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 $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g 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 $P_o = 0.5\ K_o\ \gamma\ H^2$, 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 Structure

Pavement Design

For design purposes, the pavement structure presented in the following tables could be used for the design of the pavement structure for the car only parking areas and access lanes.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 3 - Recommended Pavement Structure - Access Lanes and Heavy Truck Parking 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 fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

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.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the SPMDD with suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is expected that the building footprint will occupy majority of the subject site. It is expected that insufficient room will be available for exterior backfill along these walls and, therefore, the foundation wall will be blind poured against a waterproofing and drainage system placed over the shoring face.

Since the founding level of the proposed structure will be located below the expected high groundwater level, consideration may be given to installing a groundwater infiltration suppression system to control the final groundwater infiltration volumes.

By waterproofing the vertical excavation walls and ensuring that the system continues horizontally below the perimeter footings, it will be possible to lessen the groundwater volumes entering the excavation. This can be accomplished by placing a waterproofing membrane layer against the shoring surface. The membrane should start from the bottom of the excavation when pouring the perimeter strip footings. The waterproofing membrane should extend a minimum of 1 m above the long term groundwater table to the approximate geodetic elevation of 60 m. A composite drainage system extending up to the proposed finished grade should be incorporated against the waterproofing membrane to act as a protection layer and to drain any water breaching the waterproofing membrane system. A groundwater infiltration suppression system should also be provided for any elevator shaft and sump pump pits (pit bottoms and walls) located within the lowest basement level.

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

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. 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 composite drainage blanket, such as Delta Drain 6000 or equivalent.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation should be provided.

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.

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 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. The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below the groundwater level. It may be possible that in localized areas, where Type 3 soils are present, a 1.5H:1V excavation side slope may be required.

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

It is anticipated that temporary shoring is required to complete the required excavation 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 temporary shoring system may be calculated with the following parameters.

Table 4 - Soil Parameters	
Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
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 are 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 & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes when placed on soil subgrade. A minimum of 300 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on bedrock subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 99% of the material's SPMDD.

It should generally be possible to re-use the site materials above the cover material if the 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

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.

It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

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

For typical ground or surface water volumes being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register in 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.

Long-Term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the perimeter or sub-slab drainage system will be directed to the proposed buildings' sump pit. Provided that the selected groundwater infiltration control system is properly implemented and approved by Paterson at the time of construction, it is expected that groundwater flow will be low (i.e. less than 40,000 L/day) with peak periods noted after rain events. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

It is understood that three underground levels are planned for the proposed development. It is anticipated that the neighbouring buildings are founded within the glacial till layer. Therefore, based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures.

It should be noted that no issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed development.

6.6 Vibration and Monitoring Criteria for 1200 mm Watermain

It is understood based on available drawings that there is an existing 1200 mm watermain located approximately 3 m north of the property boundary. It is further understood that the invert of the 1200 mm watermain within the vicinity of the site is approximately 57.08 m. Based on the bedrock depths encountered throughout the north portion of the subject site, it is expected that the watermain is founded on bedrock. The lateral zone of influence for settlement sensitive structures founded on bedrock is 6V:1H.

The purpose of the vibration monitoring and control program provided herein is to establish acceptable vibration monitoring procedures and limits for the existing 1200 mm watermain. The vibration monitoring and control program also provides protocols in the event that vibration exceedance is measured.

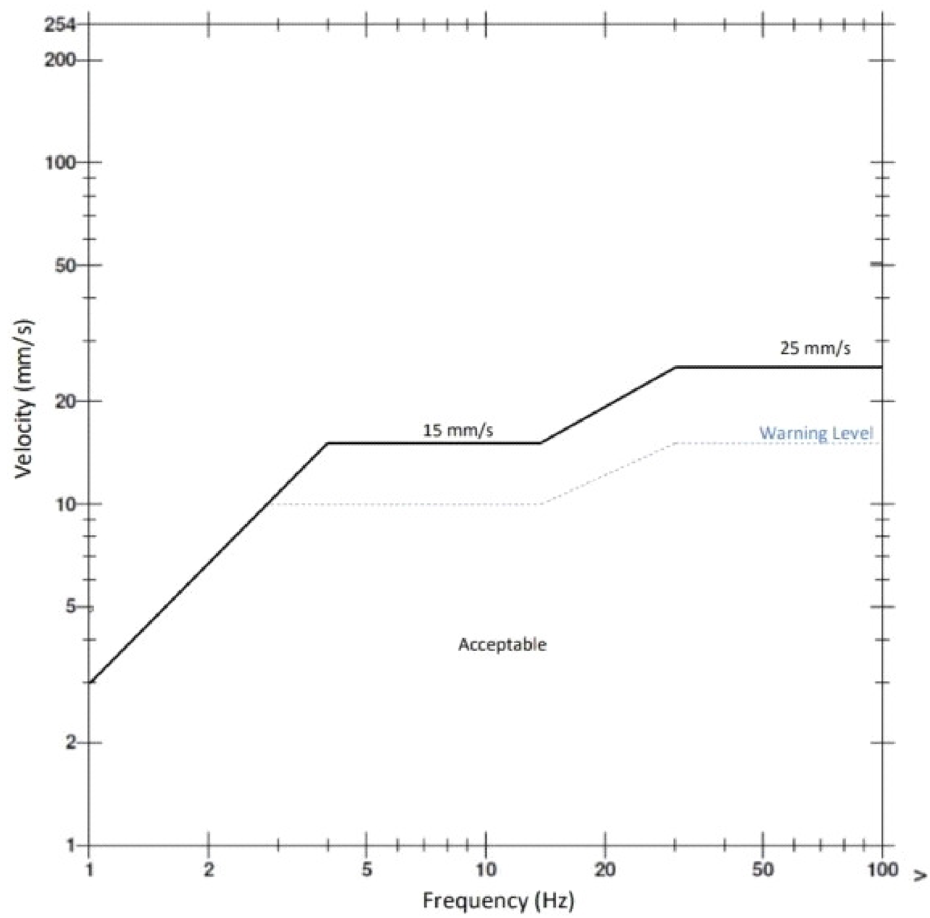
The monitoring program will incorporate real time results at the existing watermain segment adjacent to the subject site. The monitoring equipment should consist of tri-axial seismographs, capable of measuring vibration intensities up to 254 mm/s at frequencies of 2 to 250 Hz. At least four (4) vibration monitoring devices should be placed adjacent to the existing watermain, spaced equidistant apart and spanning the watermain segment for the entire length of the site boundary. It is recommended that the vibration monitoring devices be installed near the obvert level of the existing watermain and be periodically inspected during the construction program.

The VMCP, should be provided to all parties involved with the construction for review. A meeting between Paterson and the site contractor should be conducted prior to any excavation or construction of the subject site to review the following:

- ❑ Review the pre-condition/pre-construction survey;
- ❑ Control measures (i.e vibrations, noise);
- ❑ Monitoring locations;
- ❑ Tracking and reporting of excavation progress, and;
- ❑ Review procedure for exceedances (i.e vibrations, noise), complaints, evaluation and corrective measures.

When an event is triggered, Paterson will review the results and provide any necessary feedback. Otherwise, the vibration results will be summarized in a weekly report. The following figure illustrates the vibration limits for the adjacent watermain segment.

Vibration Criteria for the 1200 mm Watermain



As illustrated on the above figure, a peak particle velocity of 10 mm per second or less (measured at the watermain) for a frequency range of 4 to 12 Hz is considered acceptable. The acceptable peak particle velocity from 12 to 30 Hz must be interpolated based on the graph above. For a frequency of 30 to 100 Hz, a peak particle velocity of 15 mm per second (measured at the watermain) is considered acceptable.

Weekly reporting of the monitoring program and recommendations will be provided to the owner and the City of Ottawa. If the recommended vibration limit is exceeded, Paterson will notify the site superintendent and construction operations will be stopped.

The monitoring protocol should include the following information:

Warning Level Event

- ❑ Paterson will review all vibrations over the established warning level, illustrated by the blue line in the above figure, and;
- ❑ Paterson will notify the contractor if any vibrations occur due to construction activities and are close to exceedance level.

Exceedance Level Event

- ❑ Paterson will notify all the relevant stakeholders via email if any vibrations surpass the exceedance level, illustrated by the black line in the above figure.
- ❑ Ensure monitors are functioning
- ❑ Issue the vibration exceedance result

The data collected should include the following:

- ❑ Measured vibration levels
- ❑ Distance from the construction activity to monitoring location
- ❑ Vibration type

Monitoring should be compliant with all related regulations.

Incidence & Exceedance Reporting

In case a vibration incident/exceedance occurs from construction activities, the Senior Project Management and any relevant personnel should be notified immediately. A report should be completed which contains the following:

- ❑ Identify the location of vibration exceedance
- ❑ The date, time and nature of the exceedance/incident
- ❑ Purpose of the exceeded monitor and current vibration criteria
- ❑ Identify the likely cause of the exceedance/incident
- ❑ Describe the response action that has been completed to date
- ❑ Describe the proposed measures to address the exceedance/incident.

The contractor should implement mitigation measures for future excavation or any construction activities as necessary and provide updates on the effectiveness of the improvement. Response actions should be pre-determined prior to excavation, depending on the approach provided to protect elements. Processes and procedures should be in-place prior to completing any vibrations to identify issues and react in a quick manner in the event of an exceedance.

6.7 Winter Construction

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

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

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

6.8 Corrosion Potential and Sulphate

One (1) sample was submitted for testing. The analytical test results of the soil sample indicate that the sulphate content is less than 0.1%. These results along with the chloride and pH value are indicative that Type 10 Portland cement (Type GU) would be appropriate for this site. The chloride content and the pH of the sample indicate they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a low to moderate corrosive environment.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- ❏ Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.
- ❏ Review proposed foundation drainage design and requirements.
- ❏ Complete field reviews of the proposed groundwater infiltration suppression system for the foundation, elevator shaft and sump pump systems.
- ❏ Observation of all bearing surfaces prior to the placement of concrete.
- ❏ Sampling and testing of the concrete and fill materials used.
- ❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❏ Observation of all subgrades prior to backfilling.
- ❏ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ❏ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with Paterson's present understanding of the project. We request permission to review the recommendations when the drawings and specifications are completed.

A geotechnical 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 the 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 its 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 Dentech Holdings Inc. or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Nicole R.L. Patey, B.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- ❏ Dentech Holdings Inc. (email copy)
- ❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Residential Building
797 Richmond Road - Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5719

REMARKS

HOLE NO. BH 1-21

BORINGS BY CME-55 Low Clearance Drill **DATE** 2021 March 3

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic Concrete	0.10
FILL: Crushed stone with silty sand	0.64
FILL: Brown silty sand some gravel, clay, trace asphalt and organics	1.45
FILL: Brown silty sand some clay and gravel	1.98
Brown **SILTY CLAY** trace sand	2.29
GLACIAL TILL: Compact brown silty sand to silty clay with gravel, cobbles and boulders - Increasing clay content with depth - Grey by 5.3 m depth	5.97
End of Borehole	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	46	25
SS	3	46	7
SS	4	33	18
SS	5	58	11
SS	6	75	7
SS	7	50	9
SS	8	63	13

DEPTH (m)	ELEV. (m)
0	64.17
1	63.17
2	62.17
3	61.17
4	60.17
5	59.17

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Residential Building
797 Richmond Road - Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5719

REMARKS

HOLE NO. BH 2-21

BORINGS BY CME-55 Low Clearance Drill

DATE 2021 March 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.11
Asphaltic Concrete 0.08							
FILL: Crushed stone with brown silty sand 0.69		AU	1				
FILL: Brown silty sand with gravel, trace organics 1.52		SS	2	38	12	1	63.11
GLACIAL TILL: Compact brown silty sand to silty clay with gravel, trace cobbles and boulders		SS	3	54	16	2	62.11
		SS	4	67	40	3	61.11
		SS	5	63	16	4	60.11
		SS	6	75	33		
4.88		SS	7	67	2	5	59.11
GLACIAL TILL: Grey clayey silt with sand, gravel, cobbles and boulders 5.87		SS	8	90	+50		
BEDROCK: Good to excellent quality grey dolostone with interbedded black shale		RC	1	100	0	6	58.11
		RC	2	100	77	7	57.11
8.48		RC	3	100	97	8	56.11
End of Borehole							
(GWL @ 4.87 m depth - March 16, 2021)							

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Residential Building
797 Richmond Road - Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5719

REMARKS

HOLE NO. BH 3-21

BORINGS BY CME-55 Low Clearance Drill **DATE** 2021 March 3

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic Concrete	0.08				
FILL: Crushed stone with brown silty sand	0.53	AU	1		
FILL: Brown silty sand with crushed stone, construction debris, and organics	1.52	SS	2	79	41
FILL: Brown silty sand with gravel, trace crushed stone	2.13	SS	3	71	15
GLACIAL TILL: Compact brown silty sand to silty clay with gravel, trace cobbles and boulders		SS	4	63	16
- Grey by 3.0 m depth		SS	5	46	9
- Decreasing clay content with depth		SS	6	54	23
		SS	7	42	40
		SS	8	54	31
		SS	9	21	36
	7.11	SS	10	61	+50
BEDROCK: Fair to good quality grey dolostone with interbedded black shale		RC	1	100	46
		RC	2	100	67
	9.91	RC	3	100	78
End of Borehole					
(GWL @ 7.36 m depth - March 16, 2021)					

DEPTH (m)	ELEV. (m)
0	64.27
1	63.27
2	62.27
3	61.27
4	60.27
5	59.27
6	58.27
7	57.27
8	56.27
9	55.27

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

○ **Water Content %**

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Storey Residential Building
797 Richmond Road - Ottawa, Ontario

DATUM Geodetic

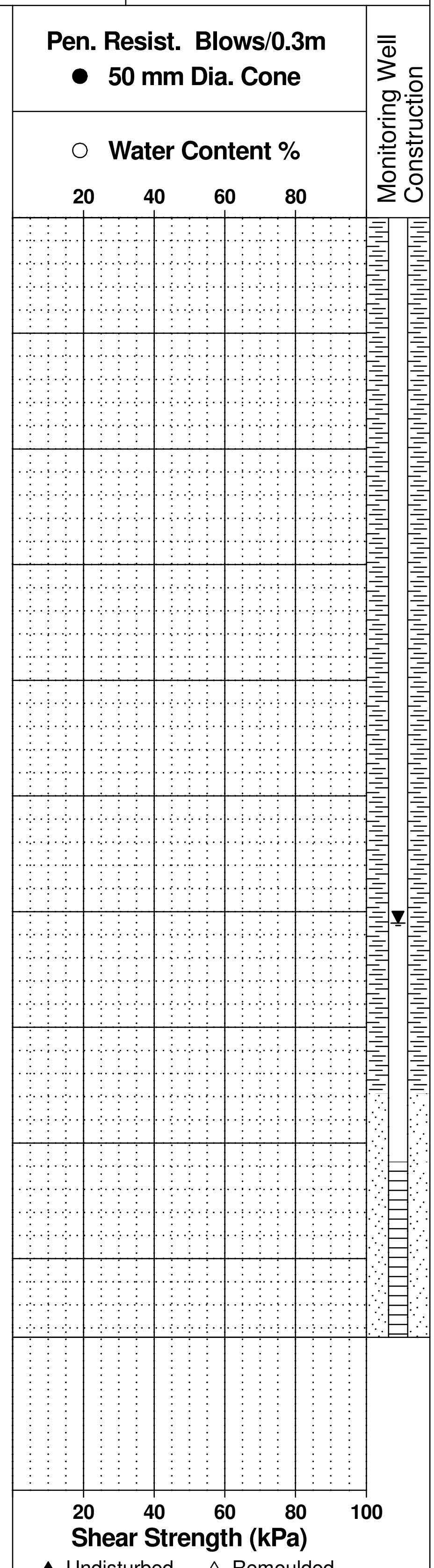
FILE NO. PG5719

REMARKS

HOLE NO. BH 4-21

BORINGS BY CME-55 Low Clearance Drill **DATE** 2021 March 4

SOIL DESCRIPTION	Depth to base of layer (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.07
Asphaltic Concrete	0.08						
FILL: Crushed stone with brown silty sand	0.66	AU	1				
FILL: Brown silty sand with gravel and brick fragments	1.45	SS	2	71	21	1	63.07
FILL: Brown silty clay with sand, some gravel, trace brick fragments and glass	2.21	SS	3	50	10	2	62.07
GLACIAL TILL: Compact brown silty sand with clay, gravel, cobbles and boulders		SS	4	42	19		
- Grey by 3.0 m depth		SS	5	54	8	3	61.07
- Decreasing clay content with depth		SS	6	50	9	4	60.07
	5.26	SS	7	46	36	5	59.07
GLACIAL TILL: Compact to dense grey silty sand with gravel, cobbles and boulders		SS	8	75	26	6	58.07
	7.34	SS	9	13	61	7	57.07
BEDROCK: Good to excellent quality grey dolostone with interbedded black shale		RC	1	100	80		
		RC	2	100	93	8	56.07
	9.68	RC	3	100	96	9	55.07
End of Borehole							
(GWL @ 6.10 m depth - March 16, 2021)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

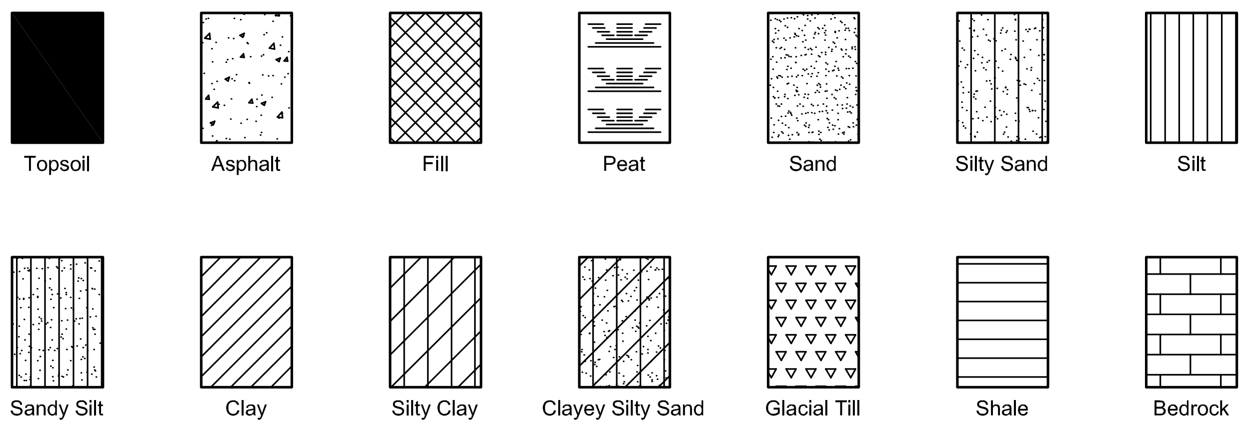
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

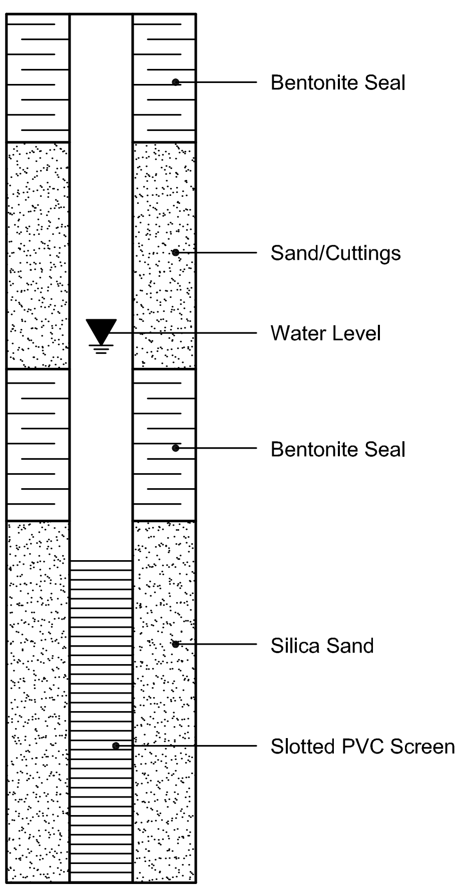
SYMBOLS AND TERMS (continued)

STRATA PLOT

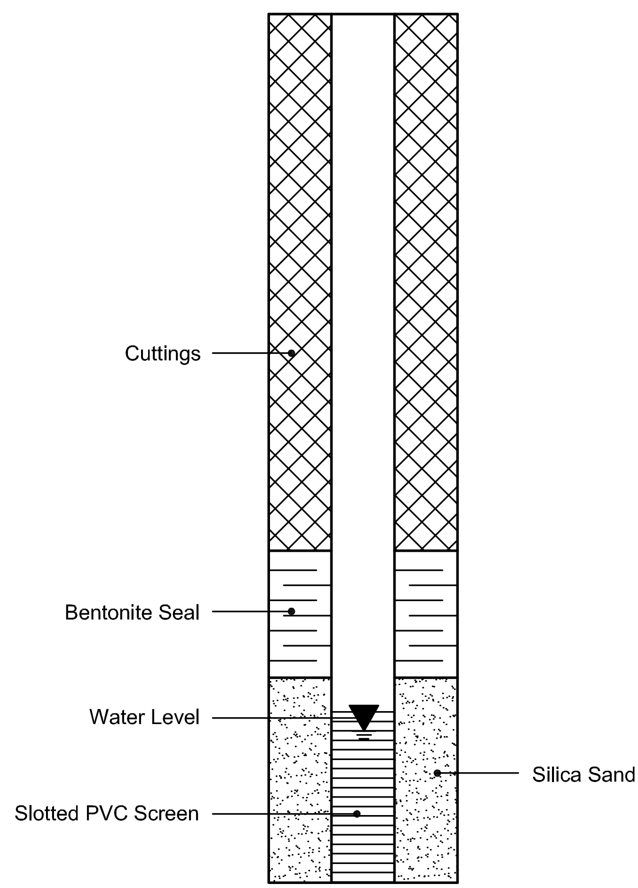


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 33000

Report Date: 13-Apr-2021

Order Date: 12-Apr-2021

Project Description: PG5719

	Client ID:	BH2-SS4	-	-	-
	Sample Date:	04-Mar-21 09:00	-	-	-
	Sample ID:	2116052-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	92.3	-	-	-
General Inorganics					
pH	0.05 pH Units	7.95 [1]	-	-	-
Resistivity	0.10 Ohm.m	41.7	-	-	-
Anions					
Chloride	5 ug/g dry	56 [1]	-	-	-
Sulphate	5 ug/g dry	53 [1]	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5719-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN

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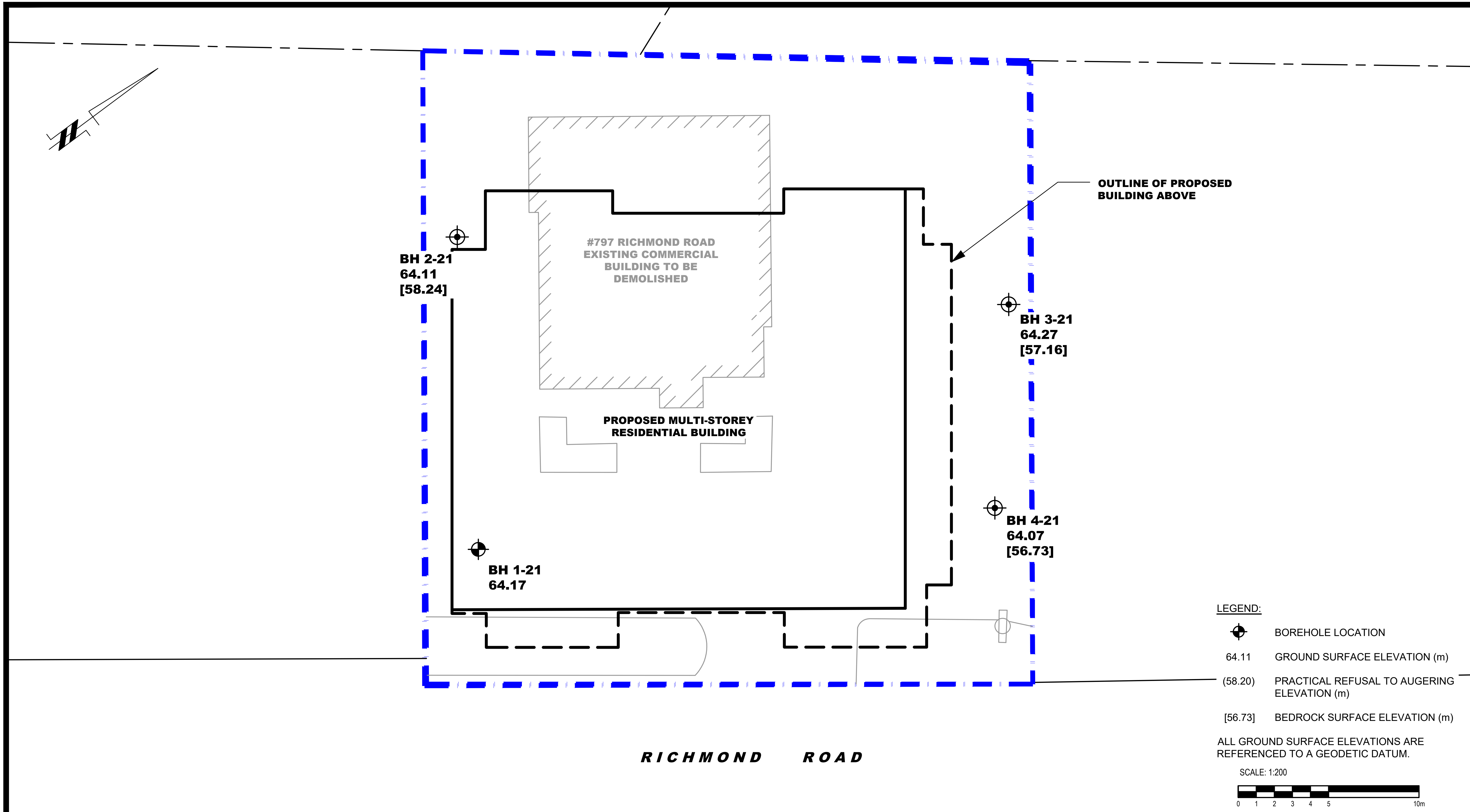

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NO.	REVISIONS	DATE	INITIAL

DENTECH HOLDINGS

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY RESIDENTIAL BUILDING
797 RICHMOND ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:200
Date: 03/2021
Drawn by: NFRV
Report No.: PG5719-1
Checked by: VD
Dwg. No.: PG5719-1
Approved by: DJG
Revision No.:

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