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**PCL Constructors Canada Inc.**
49 Auriga Drive
Ottawa, On
K2E 8A1

Attention: **Mr. David Wroblewski**

Subject: **Preliminary Geotechnical Investigation**
**Proposed Institutional Building**
**200 Lees Avenue - Ottawa, Ontario**

Sir,

Paterson Group (Paterson) was commissioned by PCL Constructor Canada Inc (PCL) to conduct a supplementary geotechnical investigation to review and supplement the information provided in “Geotechnical Report, Proposed New Building, Lees Campus”, report number 20144766 dated September 8, 2020 by Golder for the aforementioned site.

## 1.0 Proposed Project

It is of our understanding that the University of Ottawa is proposing to build a new campus building at the aforementioned site. The building is to consist of a 5 storey slab on grade structure constructed on a series of deep foundation elements (caisson and driven piles). Paterson reviewed the information provided by Golder in the above noted report and completed a supplemental field investigation to provide further geotechnical recommendations for the construction of the proposed building.

## 2.0 Field Program and Observations

### Field Program

The field program for the current investigation was carried out on between June 11 and 14, 2021 and consisted of a total of 7 boreholes sampled to a maximum depth of 17.3 m below the existing grade.

The borehole locations for the current investigations were determined in the field by Paterson personnel taking into consideration existing borehole coverage and existing site features. The locations of the boreholes are illustrated on the Test Hole Location Plans attached.

The boreholes were put down using a track-mounted auger drill. The rigs were operated by a two person crew. All fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer. The testing procedure for boreholes consisted of augering to the required depths at the selected locations and sampling the overburden.

**Surface Conditions**

The subject site is currently occupied by the existing campus owned by the University of Ottawa. The site is fairly flat and at grade with the neighbouring road way. The LRT tracks and Lees Station are located directly west of the site and lowered within the independent corridor. The Rideau River circles the site along the south and east side of the site and Highway 417 and Lees Avenue are located along the north portion of the property.

The existing campus consist of 5 buildings linked by covered and structural walkways. A sports field is location in the eastern portion of the site. It is understood that building B, C and D will be demolished as part of this project. The remainder of the site consist of parking area with access lanes with a green space between building D and A.

The banks of the Rideau river are sloped down approximately 10 to 12 m down. A slope stability analysis was completed by Golder to review the proposed development setbacks.

**Subsurface Conditions**

Based on available information and the completed field investigation the subsurface conditions at the available borehole locations consists of a fill layer, consisting mainly of silt/silty sand with cinder and ash. The fill layer was noted to be in a loose to dense state of compaction. A glacial till consisting of sand, cobbles and boulders within a clayey silt soil matrix was encountered underlying the fill layer. A shale bedrock was encountered underlying the glacial till layer.

**Groundwater**

Historical groundwater level readings were recorded at the borehole locations. The groundwater level readings indicate that the longterm groundwater table is located approximately 7 to 8 m below existing grade.

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. It should be noted that groundwater levels are subject to seasonal fluctuations, therefore the groundwater levels could vary at the time of construction.

**Laboratory Testing**

he soil samples recovered from our field investigation were examined in our laboratory to collaborate the field findings. Three representative bedrock sample were tested under unconfined compression strength.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 testing results are attached for reference.

# 3.0 Geotechnical Assessment

## 3.1 Site Grading and Preparation

**Stripping Depth**

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

**Fill Placement**

Fill used for grading beneath the building footprint, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It 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 building area should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

## 3.2 Foundation Design

### Conventional Shallow Foundation

It is expected that auxiliary structures can be founded on conventional shallow footings. For areas where a fill layer is encountered at the underside of footing, it is recommended to sub-excavate 600 mm below the underside of footing and reinstate with a select subgrade material, such as OPSS Granular B Type II, in maximum 300 mm loose lifts and compacted to a minimum 98% of its SPMDD.

It is recommended that a proof-rolling program be completed by a vibratory roller making several passes and approved by Paterson personnel prior to placement of the select subgrade material. Any poor performing areas noted during the proof-rolling program should be removed and reinstated with a select subgrade fill compacted to 98% of its SPMDD under dry and above freezing temperatures.

Footings on a compacted engineered fill placed over the approved fill layer can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **200 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

### 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 silt/silty sand above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

### Foundation Option - End Bearing Piled Foundation

A deep foundation system driven to refusal in the bedrock is recommended for foundation support of the proposed building. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance values at ultimate limit states (ULS) are given in Table 1. A resistance factor of 0.4 has been incorporated into the factored at ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

**Table 1 - Pile Foundation Design Data**

| Pile Outside Diameter (mm) | Pile Wall Thickness (mm) | Geotechnical Axial Resistance | Geotechnical Uplift Resistance |
|---|---|---|---|
| | | Factored at ULS (kN) | Factored at ULS (kN) (assumed 12 m pile) |
| 245 | 9 | 1350 | 200 |
| 245 | 11 | 1425 | 200 |
| 245 | 13 | 1500 | 200 |

The minimum centre-to-centre pile spacing is 2.5 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

It is expected that the existing piles will be subexcavated and will not carry any loads from the new proposed building. The new pile foundations should be installed leaving a minimum spacing of 300 mm between the edge of the existing pile and new pile.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.

The structural axial capacity of the pile is governed by its structural strength at the neutral plane when subjected to the permanent load plus the downdrag load. Transient live load is not to be included. At or below the pile cap, the structural strength of the embedded pile is determined as a short column subjected to the permanent load plus the transient live load, but downdrag load is to be excluded.

At the depth of the neutral plane where the downdrag load is applied, the pile structure is well confined. The 4th edition of the Canadian Foundation Engineering Manual recommends that the allowable structural axial capacity of piles at the neutral plane, for resisting permanent load plus the downdrag load, can be determined by applying a factor of safety of 1.5 to the pile material strength (steel yield and concrete 28 day compressive strength).

## Foundation Option - Drilled Shafts and Caissons

End bearing cast-in-place caissons can be used where supplemental axial resistance is required for structural design for the proposed building. The caisson should be installed by driving a temporary steel casing and excavating the soil through the casing. A minimum of 35 MPa concrete should be used to in fill the caissons. The caissons are to be structurally reinforced over their entire length.

Two conditions for drilled shafts are applicable for this site. The first alternative is a caisson installed on the sound bedrock augering through the weathered bedrock (end bearing). The compressive resistance for such piles is directly related to the compressive strength of the bedrock. It is recommended that the entire capacity be derived from the end bearing capacity.

The second alternative is a concrete caisson socketed into bedrock. The axial capacity is increased by the shear capacity of the concrete/rock interface. Furthermore, the tensile resistance of the caisson is increased by the rock capacity. It should be noted that the rock socket should be reinforced.

Table 2 below presents the estimated capacity for different typical caisson sizes for a rock bearing caisson and rock socketed caisson extending 3 m into sound bedrock.

**Table 2 - Caisson Pile Capacities**

| Caisson Diameter | | Axial Capacity (kN) | | Factored Capacity Tension at ULS (kN) | | Lateral Capacity (KN) |
|---|---|---|---|---|---|---|
| inch | mm | End Bearing | Rock Socket | End Bearing | Rock Socket | |
| 36 | 900 | 10000 | 14500 | 920 | 2700 | 800 |
| 42 | 1000 | 15000 | 19000 | 1050 | 3450 | 900 |
| 48 | 1200 | 19000 | 24500 | 1200 | 4500 | 1100 |
| 54 | 1375 | 24000 | 31000 | 1350 | 5300 | 1400 |
| 60 | 1500 | 30000 | 38000 | 1500 | 6000 | 1600 |

**notes:**
- 3 m rock socket in sound bedrock
- Reinforced caisson and rock socket when applicable
- 0.4 geotechnical factor applied to the shaft capacity

Based on the recent field investigation and bedrock coring it is expected that a weathered layer of bedrock will need to be removed to reach a sound surface. The thickness of the weathered layer was evaluated to vary from1.5 to 1.8 m across the site. It is expected that the deep foundation rig will be able to auger through the weathered bedrock layer.

Caisson lateral capacities have been provided assuming a minimum reinforcement ratio of 0.2% and inclusion of shear reinforcement consisting of reinforcement rings place 250 to 300 mm apart along the pile length. An increase lateral capacity can be achieved by increasing the reinforcement ratio and the position of the shear reinforcement. This will increase the general stiffness of the element. The caisson designer should review design loads to provide sufficient resistance to the proposed caissons.

## 3.3 Lateral Load Resistance

Lateral loads on the foundations can be resisted using passive resistance on the sides of the foundations. For Limit States Design, the resistance factor to be applied to the ultimate lateral resistance, including passive pressure, is 0.50. The total lateral resistance will be comprised of the individual contributions from up to several material layers, as follows.

Geotechnical parameters for the silty sand fill, glaical till and for typical backfill materials compacted to 98% of SPMDD in 300 mm lift thicknesses are provided in Table 3, below, along with the associated earth pressure coefficients for horizontal resistance calculations. Friction factors between concrete and the various subgrade materials are also provided in Table 3, where normal loads allow them to be used.

Where granular soils and/or granular backfill materials are present, the passive pressure can be calculated using a triangular distribution equal to $K_P \cdot \gamma \cdot H$ where:

$K_P$ = passive earth pressure coefficient of the applicable retained soil
$\gamma$ = unit weight of the fill of the applicable retained soil ($kN/m^3$)
H = height of the equivalent wall or footing side (m)

Note that for cases where the depth to the top of the structure (i.e. footing) pushing against the soil does not exceed 50% of the depth to the base of the structure, the effective value of H in the above noted relationship will be the overall depth to the base of the structure. There will also be “edge effects” where the effective width of soil providing the resistance can be increased by 50% of the effective depth on each side of the pushing structural component.

Note that where the foundation extends below the groundwater level, the effective unit weight should be utilized for the saturated portion of the soil or fill.

Should additional passive resistance be require, the horizontal component of the axial resistance of battered piles (up to 1H:3V inclination), or anchors can be used in the building foundation design.

**Table 3 - Geotechnical Parameters for Uplift and Lateral Resistance Design**

| Material Description | Unit Weight (kN/m$^3$) Drained $\gamma_{dr}$ | Unit Weight (kN/m$^3$) Effective $\gamma'$ | Internal Friction Angle (°) $\varphi'$ | Friction Factor, tan δ | Earth Pressure Coefficients Active $K_A$ | Earth Pressure Coefficients At-Rest $K_O$ | Earth Pressure Coefficients Passive $K_P$ |
|---|---|---|---|---|---|---|---|
| OPSS Granular A Fill (Crushed Stone) | 22.0 | 13.7 | 38 | 0.55 | 0.22 | 0.36 | 4.2 |
| OPSS Granular B Type I Fill (Well-Graded Sand-Gravel) | 21.5 | 13.4 | 36 | 0.45 | 0.26 | 0.41 | 3.9 |
| OPSS Granular B Type II Fill (Crushed Stone) | 22.5 | 14.0 | 40 | 0.55 | 0.20 | 0.33 | 4.6 |
| Glacial Till | 22.0 | 13.5 | 35 | 0.40 | 0.27 | 0.42 | 3.7 |
| In Situ Silty Sand or Site Excavated Silty Sand Fill | 18.0 | 11.2 | 32 | 0.40 | 0.30 | 0.46 | 3.3 |

**Notes:**
1. Properties for backfill materials are for condition of 98% of standard Proctor maximum dry density.
2. The earth pressure coefficients provided are for horizontal profile.

## 3.4 Design for Earthquakes

The results of seismic shear wave velocity testing performed by others indicated an average shear wave velocity, $Vs_{30}$, at this site of 467 m/s. A **Site Class C** is therefore applicable for design across the site. 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.

# 4.0 Design and Construction Precautions

## 4.1 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided for footings supported on an undisturbed, compact silty sand to sand bearing surface. However, perimeter footings supported directly on clean, surface-sounded bedrock will only require 0.6 m of soil cover for frost protection.

Exterior unheated footings, such as isolated exterior 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 a combination of soil cover and foundation insulation.

## 4.2 Excavation Side Slopes

The side slopes of excavations in the overburden soils should be sloped back at acceptable slopes from the start of the excavation until the structure is backfilled. The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly 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 Paterson 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.

## 4.3 Rock Anchor Design

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor. It should be noted that interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each anchor taken individually.

A third failure mode of shear failure along the grout/steel interface should also bereviewed by a qualified structural engineer to ensure all typical failure modes have been reviewed. Typical rock anchor suppliers, such as Dywidag Systems International (DSI Canada), have qualified personnel on staff to recommend appropriate rock anchor size and materials.

It should be further noted that center to center spacing between bond lengths be at least four times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with post-tensioned load or not prior to being put into service. To resist seismic uplift pressures, a passive rock anchor system can be used. It should be noted that a post-tensioned anchor will take the uplift load with much less deflection than a passive anchor.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity, as well an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. As the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cementitious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break.

**Grout to Rock Bond**

Based on the type of rock encountered on site, a factored tensile grout to rock bond resistance value at ULS of **1 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 30 MPa is recommended.

**Rock Cone Uplift**

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 58** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.608 and 0.00198**, respectively.

**Recommended Rock Anchor Lengths**

Parameters used to calculate rock anchor lengths are provided in Table 4.

**Table 4 - Parameters used in Rock Anchor Review**

| | |
|---|---|
| Grout to Rock Bond Strength - Factored at ULS | 1.0 MPa |
| Compressive Strength - Grout | 30 MPa |
| Rock Mass Rating (RMR) - Fair Good quality shale<br>Hoek and Brown parameters | 58<br>m=0.608 and s=0.00198 |
| Unconfined compressive strength - Shale bedrock | 50 MPa |
| Unit weight - Submerged Bedrock | 15.2 kN/m$^3$ |
| Apex angle of failure cone | 60$^o$ |
| Apex of failure cone | mid-point of fixed anchor length |

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 to 250 mm diameter hole are provided in Table 5 below.

The factored tensile resistance value have been calculated using the Hoek and Brown design parameters and the design method presented by Serrano and Olalla (1999).

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a tremie tube to place grout from the bottom up in the anchor holes is further recommended.

**Table 5 - Recommended Rock Anchor Lengths - Grouted Rock Anchor**

| Diameter of Corehole (mm) | Anchor Lengths (m) | | | Factored Tensile Resistance (kN) |
|---|---|---|---|---|
| | Bonded Length | Unbonded Length | Total Length | |
| 75 | 1.6 | 1.0 | 2.6 | 350 |
| | 2.4 | 1.0 | 3.4 | 550 |
| | 4.3 | 0.6 | 4.8 | 1000 |
| | 6.5 | 1.0 | 7.5 | 1500 |
| 125 | 1.4 | 1.0 | 2.4 | 350 |
| | 1.6 | 1.2 | 2.8 | 550 |
| | 2.8 | 1.4 | 4.2 | 1000 |
| | 4.0 | 1.5 | 5.5 | 1500 |
| 150 | 2.4 | 1.8 | 4.2 | 1000 |
| | 3.3 | 2.0 | 5.3 | 1500 |
| | 4.5 | 2.0 | 6.5 | 2000 |
| 250 | 2.5 | 2.5 | 5.0 | 1500 |
| | 3.3 | 3.0 | 6.3 | 2000 |
| | 4.0 | 2.9 | 6.9 | 2500 |

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

The provided rock anchor parameters can be used in conjunction with the preliminary pile foundation design parameters for the design of micropile if supplemental tensile resistance is required.

**Installation Procedures**

Rock anchor should be installed using a down the hole rotary air hammer. The operator should keep a log of all drill holes. A casing should be used the advanced the drilling tool through the existing fill material and glacial till layer.

Once completed the hole should be flushed through the casing until the water returns clear. It will be important to remove rock dust and particles prior to the placement of the central reenforcement rod and grouting.

The anchor should be grouted by gravity using a tremie tube extending all the way to the bottom of the hole. Pressure grouting can be used if supplemental capacity is required.

## 4.4 Slab on Grade Construction

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the fill layer will required to be proofed rolled using an oversized compactor making several passes.

It is also expected that a crane/foundation rig working platform will be put in place. The working platform will provide a suitable surface to begin backfill below the slab on grade. The platform should be stripped of deleterious material and proof rolled prior to backfilling for the slab on grade.

The slab on grade can be backfilled using approved site excavated material free of deleterious material and place in under dry conditions and above freezing temperatures. All backfill material required to raise the grade within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

For mechanical and electrical conduit, it is recommended that the upper 450 mm of sub-floor fill consist of OPSS Granular A crushed stone. If site excavated material is used to backfill below the slab on grade, the upper granular layer and fill layer should be separated with a non woven geotextile such as Terrafix 420R or equivalent.

Paterson should review the proof rolling activities as well as all backfilling activities. Paterson will conduct regular inspection to test density, compaction and review bearing surface conditions.

## 4.5 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 bedding for sewer and water pipes when placed on soil 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 (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

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

## 4.6 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of car only parking areas, local streets and roadways with bus traffic. It should be noted that for car only parking areas, an Ontario Traffic Category A is applicable. For local roadways and roadways with bus traffic, an Ontario Traffic Category B and Category D should be used for design purposes, respectively.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

**Table 6 - Recommended Pavement Structure - 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 silty clay or sand or crushed stone material placed over in situ soil. | |

| Table 7 - Recommended Pavement Structure - Local Roadways, Access Lanes and Heavy Vehicle Parking | |
|---|---|
| **Thickness (mm)** | **Material Description** |
| 40 | **Wear Course -** Superpave 12.5 Asphaltic Concrete |
| 50 | **Binder Course -** Superpave 19.0 Asphaltic Concrete |
| 150 | **BASE -** OPSS Granular A Crushed Stone |
| 450 | **SUBBASE -** OPSS Granular B Type II |
| **SUBGRADE** - Either fill, in situ silty clay or sand or crushed stone material placed over in situ soil. | |

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for parking areas and local roadways and access lanes. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

The proposed pavement structure, where it abuts the existing pavement, should match the existing pavement layers. It is recommended that a 300 mm wide and 50 mm deep stepped joint be provided where the new asphalt layer joins with the existing asphalt layer to provide more resistance to cracking at the joint.

## 4.7 Corrosion Protection

The results of previous and current corrosion testing are indicative of a moderate to aggressive environment to corrosion and exposed ferrous. The result also indicate that the sulphate content is below or marginally above 0.1% within the lower layer of fill. The exceedance is not significant from a geotechnical perspective and has been found to be localized to some pockets around the site. It is recommended that a Type GU or GUL Portland cement (normal cement) be considered for this site.

The proposed steel pipe piles should be designed with a minimum sacrificial steel layer of 2 mm. The above noted capacities take into consideration the effect of long term corrosion. No further steel corrosion protection will be required for the proposed steel piles.

If the option to reused site excavated fill to backfill below the slab on grade is considered, it is further recommended that separation layer be placed between the fill, pile caps and grade beams. The protective layer should be composed of clean important OPSS Granular A or Granular B Type I or Type II. A minimum width of 300 mm is recommended to separate the structural element and site excavated fill material.

## 4.8 Slope Stability Review

Paterson completed a review of the slope stability analysis and recommended limit of hazard land recommended by Golder. Two slope sections closest to the proposed redevelopment have been analysed. Section A is located in the south west corner of the site and section B is located south of the existing Building D. Reference should be made to the report noted above for further information.

Based on the finding of the analysis a limit of hazard lands setback of 13 m for section A and 10 m for Section B is recommended from the top of the existing slope.

Paterson completed a field review of the slope in the area during the current field program. The slope was noted to be heavily vegetation and no erosion was observed along the site. Erosion protection was also noted in some area. The erosion control measure was noted to consist of rip rap stone placed along the bottom of the slope.

Based on our field review and current geotechnical information acquired from site, the proposed setback are acceptable for the proposed project. It is expected that the building will be constructed on a deep foundation system and will not impact the stability of the existing slopes.

## 5.0 Recommendations

It is recommended that the following be completed once the master plan and site development are determined:

❑ Observation of all bearing surfaces prior to the placement of concrete.

❑ Conduct a full time geotechnical inspection program during the piling activities.

❑ Complete a full material and testing inspection program during the construction of the project.

❑ 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 placing backfilling materials.

❑ Field density tests to ensure that the specified level of compaction has been 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 Paterson's recommendations could be issued upon request, following the completion of a satisfactory material testing and observation program by the geotechnical consultant.

## 6.0 Statement of Limitations

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

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only and test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from the test locations, Paterson requests immediate notification to permit reassessment of the recommendations.

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 PCL Constructors Canada Inc., or their agents, is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

We trust this report meets your present requirements.

Best Regards,

**Paterson Group Inc.**

LICENSED PROFESSIONAL ENGINEER
October 17, 2021
J. R. VILLENEUVE
100504344
PROVINCE OF ONTARIO

Joey R. Villeneuve, M.A.Sc, P.Eng.

David J. Gilbert, P.Eng

**Attachments**

- ☐ Soil Profile and Test Data Sheets
- ☐ Symbols and Terms
- ☐ Laboratory Testing Results
- ☐ Analytical Testing Results
- ☐ Figure 1 - Key Plan
- ☐ Drawing PG5656-01 - Test Hole Location Plan

**patersongroup** Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**uOttawa Faculty of Health Sciences Building**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

**BORINGS BY** CME-55 Low Clearance Drill

**DATE** June 14, 2021

**FILE NO.** PG5656

**HOLE NO.** BH 1-21

| 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<br>○ Water Content %<br>20 40 60 80 | Piezometer Construction |
|---|---|---|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | | 0 | 61.95 | | |
| Asphaltic concrete 0.10 | | AU | 1 | | | | | | |
| **FILL:** Brown silty sand with crushed stone and gravel<br>- trace asphalt by 0.6m depth 1.45 | | SS | 2 | 40 | 27 | 1 | 60.95 | | |
| **FILL:** Dark brown silty sand with ash and coal 2.21 | | SS | 3 | 75 | 19 | 2 | 59.95 | | |
| **FILL:** Brown to grey silty clay with topsoil, some ash and coal 2.97 | | SS | 4 | 50 | 5 | 3 | 58.95 | | ▽ |
| **GLACIAL TILL:** Very stiff, brown silty clay with sand, gravel, cobbles and boulders | | SS | 5 | 75 | 5 | | | | |
| | | SS | 6 | 42 | 8 | 4 | 57.95 | | |
| - grey by 4.3m depth 5.18 | | SS | 7 | 0 | 11 | 5 | 56.95 | | |
| End of Borehole<br>(GWL @ 3.0m depth based on field observations) | | | | | | | | | |

20 40 60 80 100

**Shear Strength (kPa)**

▲ Undisturbed △ Remoulded

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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**uOttawa Faculty of Health Sciences Building**
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**DATUM** Geodetic

**REMARKS**

**BORINGS BY** CME-55 Low Clearance Drill

**DATE** June 14, 2021

**FILE NO.** PG5656

**HOLE NO.** BH 2-21

| 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<br>○ Water Content %<br>20 40 60 80 | Piezometer Construction |
|---|---|---|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | | 0 | 61.94 | | |
| Asphaltic concrete 0.08 | | AU | 1 | | | | | | |
| **FILL:** Brown silty sand with crushed stone and gravel 0.60 | | SS | 2 | 62 | 9 | 1 | 60.94 | | |
| **FILL:** Ash and coal with brown silty sand, some gravel | | SS | 3 | 50 | 6 | 2 | 59.94 | | |
| | | SS | 4 | 42 | 4 | 3 | 58.94 | | |
| 3.96 | | SS | 5 | 62 | 1 | | | | |
| **TOPSOIL** 4.42 | | SS | 6 | 42 | 5 | 4 | 57.94 | | ∇ |
| Stiff, brown **CLAYEY SILT** 5.18 | | SS | 7 | 62 | 5 | 5 | 56.94 | | |
| End of Borehole | | | | | | | | | |

20 40 60 80 100

**Shear Strength (kPa)**

▲ Undisturbed △ Remoulded

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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**uOttawa Faculty of Health Sciences Building**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

**BORINGS BY** CME-55 Low Clearance Drill

**DATE** June 14, 2021

**FILE NO.** PG5656

**HOLE NO.** BH 3-21

| SOIL DESCRIPTION | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD | DEPTH (m) | ELEV. (m) | Pen. Resist. Blows/0.3m<br>● 50 mm Dia. Cone<br>○ Water Content %<br>20 40 60 80 | Piezometer Construction |
|---|---|---|---|---|---|---|---|---|---|
| **GROUND SURFACE** | | | | | | 0 | 61.70 | | |
| Asphaltic concrete 0.08 | ▨ | AU | 1 | | | | | | |
| **FILL:** Brown silty sand with crushed stone and gravel 0.60 | ▨ | SS | 2 | 67 | 28 | 1 | 60.70 | | |
| **FILL:** Brown silty sand, some ash and coal, trace clay and gravel | ▨ | SS | 3 | 21 | 11 | 2 | 59.70 | | |
| | ▨ | SS | 4 | 75 | 12 | | | | |
| 3.17 | ▨ | SS | 5 | 0 | 50+ | 3 | 58.70 | | |
| End of Borehole<br>Practical refusal to augering at 3.17m depth.<br>(BH dry upon completion) | | | | | | | | | |

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

# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**uOttawa Faculty of Health Sciences Building**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

**BORINGS BY** CME-55 Low Clearance Drill

**DATE** June 14, 2021

**FILE NO.** PG5656

**HOLE NO.** BH 4-21

| SOIL DESCRIPTION | Depth to base (m) |
|---|---|
| GROUND SURFACE | |
| Asphaltic concrete | 0.08 |
| **FILL:** Brown silty sand with crushed stone and gravel, trace clay | 0.60 |
| **FILL:** Dark brown to black silty sand with ash and coal, some gravel | 4.42 |
| Very stiff, grey **CLAYEY SILT,** tarce topsoil | 5.18 |
| Compact, brown **CLAYEY SILT to SANDY SILT** | 7.77 |
| Compact to very dense, grey **SILTY SAND**<br>- trace to some gravel by 10.8m depth | 14.10 |
| **BEDROCK:** Fair to excellent quality, black shale | 17.20 |
| End of Borehole<br>(GWL @ 5.5m depth based on field observations) | |

| SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|
| AU | 1 | | |
| SS | 2 | 50 | 15 |
| SS | 3 | 75 | 14 |
| SS | 4 | 17 | 5 |
| SS | 5 | 25 | 4 |
| SS | 6 | 58 | 3 |
| SS | 7 | 75 | 11 |
| SS | 8 | 67 | 21 |
| SS | 9 | 0 | 16 |
| SS | 10 | 83 | 43 |
| SS | 11 | 67 | 94 |
| SS | 12 | 75 | 78 |
| SS | 13 | 100 | 50+ |
| RC | 1 | 100 | 50 |
| RC | 2 | 100 | 73 |
| RC | 3 | 100 | 100 |

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 61.84 |
| 1 | 60.84 |
| 2 | 59.84 |
| 3 | 58.84 |
| 4 | 57.84 |
| 5 | 56.84 |
| 6 | 55.84 |
| 7 | 54.84 |
| 8 | 53.84 |
| 9 | 52.84 |
| 10 | 51.84 |
| 11 | 50.84 |
| 12 | 49.84 |
| 13 | 48.84 |
| 14 | 47.84 |
| 15 | 46.84 |
| 16 | 45.84 |
| 17 | 44.84 |

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

**patersongroup** Consulting Engineers

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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**uOttawa Faculty of Health Sciences Building**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

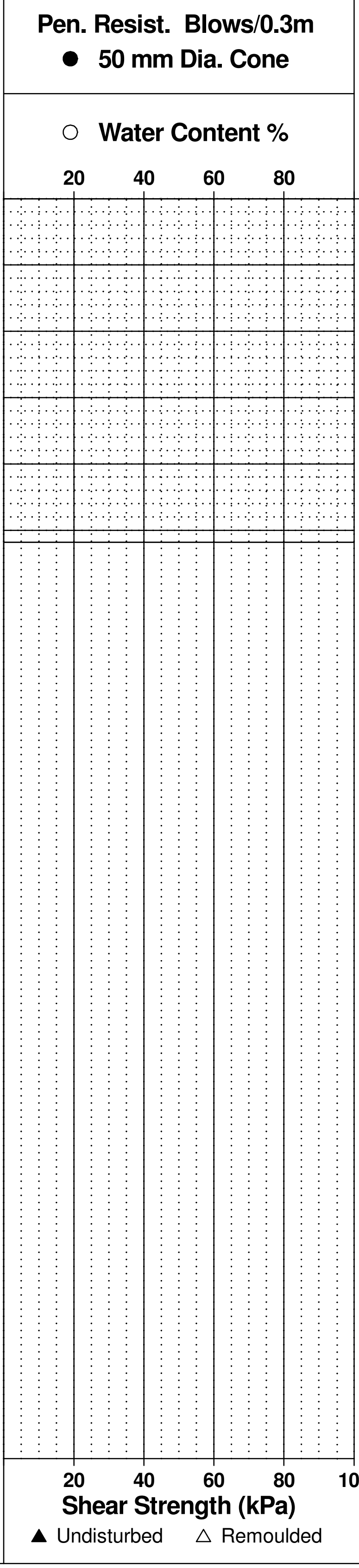
**BORINGS BY** CME-55 Low Clearance Drill

**DATE** June 11, 2021

**FILE NO.** PG5656

**HOLE NO.** BH 5-21

| SOIL DESCRIPTION<br>GROUND SURFACE | | STRATA PLOT | SAMPLE TYPE | SAMPLE NUMBER | SAMPLE % RECOVERY | SAMPLE N VALUE or RQD | DEPTH (m) | ELEV. (m) | Piezometer Construction |
|---|---|---|---|---|---|---|---|---|---|
| Asphaltic concrete | 0.08 | | AU | 1 | | | 0 | 61.95 | |
| **FILL:** Brown silty sand with crushed stone, gravel, trace wood | 0.60 | | SS | 2 | 42 | 45 | 1 | 60.95 | |
| | | | SS | 3 | 0 | | 2 | 59.95 | |
| **FILL:** Dark brown to black silty sand, some ash, coal, wood and brick, trace gravel | | | SS | 4 | 33 | 2 | 3 | 58.95 | |
| | | | SS | 5 | 25 | 10 | | | |
| | | | SS | 6 | 42 | 2 | 4 | 57.95 | |
| - some clay by 4.7m depth | 5.18 | | SS | 7 | 42 | 3 | 5 | 56.95 | |
| End of Borehole<br>(BH dry upon completion) | | | | | | | | | |



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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**uOttawa Faculty of Health Sciences Building**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

**BORINGS BY** CME-55 Low Clearance Drill

**DATE** June 11, 2021

**FILE NO.** PG5656

**HOLE NO.** BH 6-21

| SOIL DESCRIPTION | TYPE | NUMBER | % RECOVERY | N VALUE or RQD |
|---|---|---|---|---|
| **GROUND SURFACE** | | | | |
| **FILL:** Organics with silty sand 0.08 | AU | 1 | | |
| **FILL:** Brown silty sand, trace gravel, cobbles, organics 0.43 | SS | 2 | 12 | 2 |
| **FILL:** Brown to black silty sand with ash, coal, slag, trace wood, brick and gravel | SS | 3 | 0 | 3 |
| | SS | 4 | 58 | 3 |
| | SS | 5 | 33 | 5 |
| | SS | 6 | 42 | 4 |
| 5.79 | SS | 7 | 33 | 4 |
| Compact, brownt o grey **CLAYEY SILT to SANDY SILT,** trace organics 7.16 | SS | 8 | 58 | 11 |
| **GLACIAL TILL:** Dense to very dense, dark grey silty sand with gravel, cobbles and boulders, trace clay | SS | 9 | 54 | 47 |
| | SS | 10 | 58 | 26 |
| | SS | 11 | 52 | 106 |
| 12.27 | SS | 12 | 100 | 50+ |
| **BEDROCK:** Poor to excellent quality, black shale<br>- vertical seams from 12.4 to 13.5m, 13.9 to 14.2m and from 14.4 to 15.1m depths | RC | 1 | 100 | 36 |
| 15.06 | RC | 2 | 100 | 92 |
| End of Borehole<br>(GWL @ 6.10m depth based on field observations) | | | | |

| DEPTH (m) | ELEV. (m) |
|---|---|
| 0 | 62.05 |
| 1 | 61.05 |
| 2 | 60.05 |
| 3 | 59.05 |
| 4 | 58.05 |
| 5 | 57.05 |
| 6 | 56.05 |
| 7 | 55.05 |
| 8 | 54.05 |
| 9 | 53.05 |
| 10 | 52.05 |
| 11 | 51.05 |
| 12 | 50.05 |
| 13 | 49.05 |
| 14 | 48.05 |
| 15 | 47.05 |

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

○ Water Content %

20 40 60 80

Piezometer Construction

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

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# SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**
**uOttawa Faculty of Health Sciences Building**
**Ottawa, Ontario**

**DATUM** Geodetic

**REMARKS**

**BORINGS BY** CME-55 Low Clearance Drill

**DATE** June 11, 2021

**FILE NO.** PG5656

**HOLE NO.** BH 7-21

| SOIL DESCRIPTION | SAMPLE TYPE | NUMBER | % RECOVERY | N VALUE or RQD | DEPTH (m) | ELEV. (m) |
|---|---|---|---|---|---|---|
| GROUND SURFACE | | | | | 0 | 61.95 |
| **FILL:** Brown silty sand with gravel, trace cobbles and organics 0.76 | AU | 1 | | | | |
| **FILL:** Brown silty sand with ash, coal and brick, trace wood<br>- black by 3.0m depth<br>- reddish brown by 3.7m depth | SS | 2 | 33 | 2 | 1 | 60.95 |
| | SS | 3 | 21 | 1 | 2 | 59.95 |
| | SS | 4 | 50 | 5 | 3 | 58.95 |
| | SS | 5 | 50 | 3 | | |
| | SS | 6 | 33 | 3 | 4 | 57.95 |
| 4.88 | SS | 7 | 0 | 17 | 5 | 56.95 |
| Very stiff, brown to grey **SILTY CLAY,** trace organics | SS | 8 | 58 | 14 | 6 | 55.95 |
| 7.16 | | | | | 7 | 54.95 |
| **GLACIAL TILL:** Dense, dark grey silty sand with gravel, cobbles and boulders, trace clay | SS | 9 | 50 | 47 | 8 | 53.95 |
| 8.70 | | | | | 9 | 52.95 |
| **GLACIAL TILL:** Dense, dark grey silty clay with sand, gravel, cobbles, boulders and shale fragments<br>- shale fragments increasing with depth | SS | 10 | 50 | 44 | 10 | 51.95 |
| | SS | 11 | 62 | 24 | 11 | 50.95 |
| | SS | 12 | 100 | 50+ | 12 | 49.95 |
| | | | | | 13 | 48.95 |
| 14.45 | | | | | 14 | 47.95 |
| **BEDROCK:** Excellent to good quality, black shale | RC | 1 | 100 | 98 | 15 | 46.95 |
| | RC | 2 | 100 | 78 | 16 | 45.95 |
| 17.25 | | | | | 17 | 44.95 |
| End of Borehole<br>(BH 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 $p'_c$)

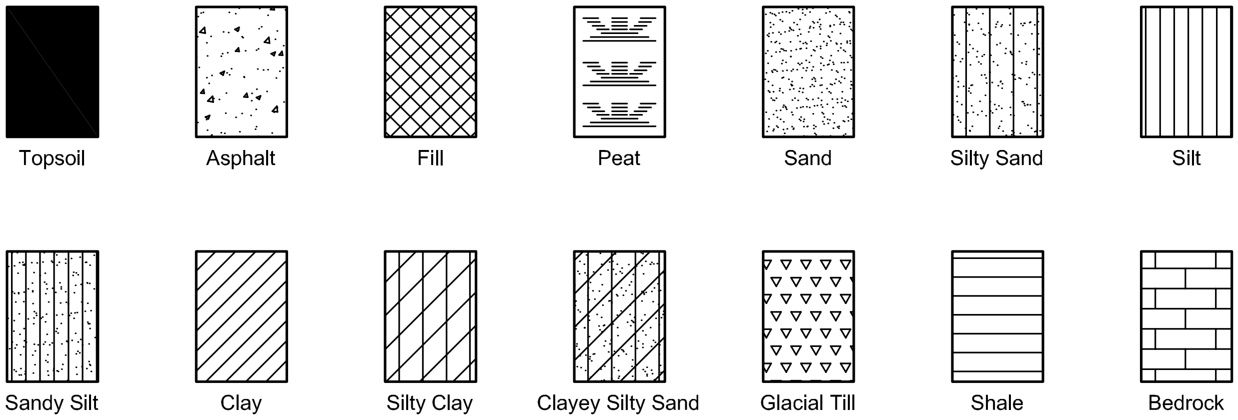
OC Ratio Overconsolidaton ratio = $p'_c / p'_o$
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

## 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.

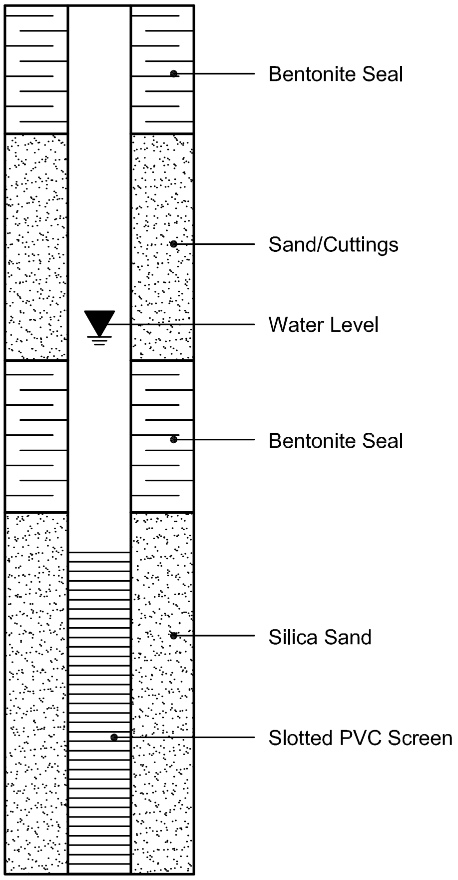
# SYMBOLS AND TERMS (continued)

## STRATA PLOT

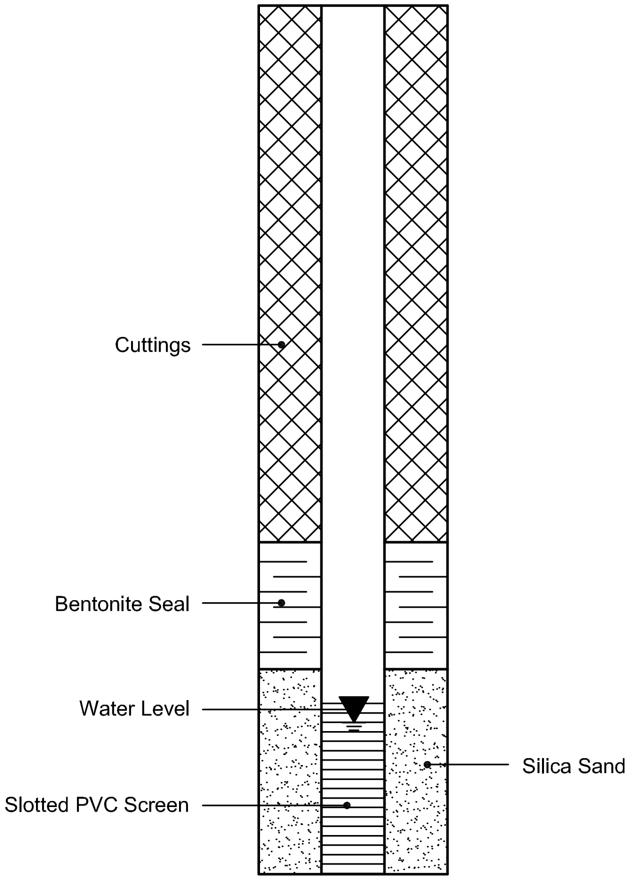


## MONITORING WELL AND PIEZOMETER CONSTRUCTION

### MONITORING WELL CONSTRUCTION



### PIEZOMETER CONSTRUCTION



patersongroup
consulting engineers

# ROCK CORE COMPRESSIVE STRENGTH ASTM D7012

| | | | |
|---|---|---|---|
| CLIENT: | **PCL c/o University of Ottawa** | | FILE No.: **PG5656** |
| PROJECT: | 200 Lees Avenue | | REPORT No.: 3 |
| SITE ADDRESS | **200 Lees Avenue** | | DATE REPT'D: 17-Jun-21 |
| STRUCTURE TYPE & LOCATION: | Bedrock | | |
| **SAMPLE INFORAMTION** | | | |
| LAB NO.: | - | - | - |
| SAMPLE NO.: | BH4 - RC2 | BH7 - RC2 | BH7 RC2 |
| LOCATION: | - | - | - |
| **SAMPLE DATES** | | | |
| DATE CAST | - | - | - |
| DATE CORED | June 11-14, 2021 | June 11-14, 2021 | June 11-14, 2021 |
| DATE RECEIVED | 17-Jun-21 | 17-Jun-21 | 17-Jun-21 |
| DATE TESTED | 17-Jun-21 | 17-Jun-21 | 17-Jun-21 |
| **SAMPLE DIMENSIONS** | | | |
| AVERAGE DIAMETER (mm) | 47.00 | 47.00 | 47.00 |
| HEIGHT (mm) | 85.00 | 72.40 | 88.20 |
| WEIGHT (g) | 420 | 400 | 400 |
| AREA ($mm^2$) | 1735 | 1735 | 1735 |
| VOLUME ($cm^3$) | 147 | 126 | 153 |
| **UNIT WEIGHT ($kg/m^3$)** | **2848** | **3184** | **2614** |
| **TEST RESULTS** | | | |
| H / D RATIO | 1.81 | 1.54 | 1.88 |
| CORRECTION FACTOR | 0.984 | 0.964 | 0.989 |
| LOAD (lbs) | 13549 | 12350 | 11797 |
| GROSS Mpa | 34.7 | 31.7 | 30.2 |
| **MPa CORRECTED** | **34.2** | **30.5** | **29.9** |
| FORM OF BREAK | I | I | I |
| DIRECTION OF LOADING | Parallel | Parallel | Parallel |
| CURING CONDITIONS | SITE→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→→ | | |
| **REMARKS** | | | |
| | | | |
| | | | |
| | | | |
| **TECHNICAL PERSONNEL** | | | |
| **TECHNICIAN: C.M.** | **VERIFIED BY:** **C. Beadow** | **APPROVED BY:** | **Joe Forsyth, P. Eng.** |
| **CERTIFIED LAB** | | | |
| John D. Paterson & Associates Ltd., 28 Concourse Gate, Nepean, ON | | | |

Certificate of Analysis

**Client: Paterson Group Consulting Engineers**

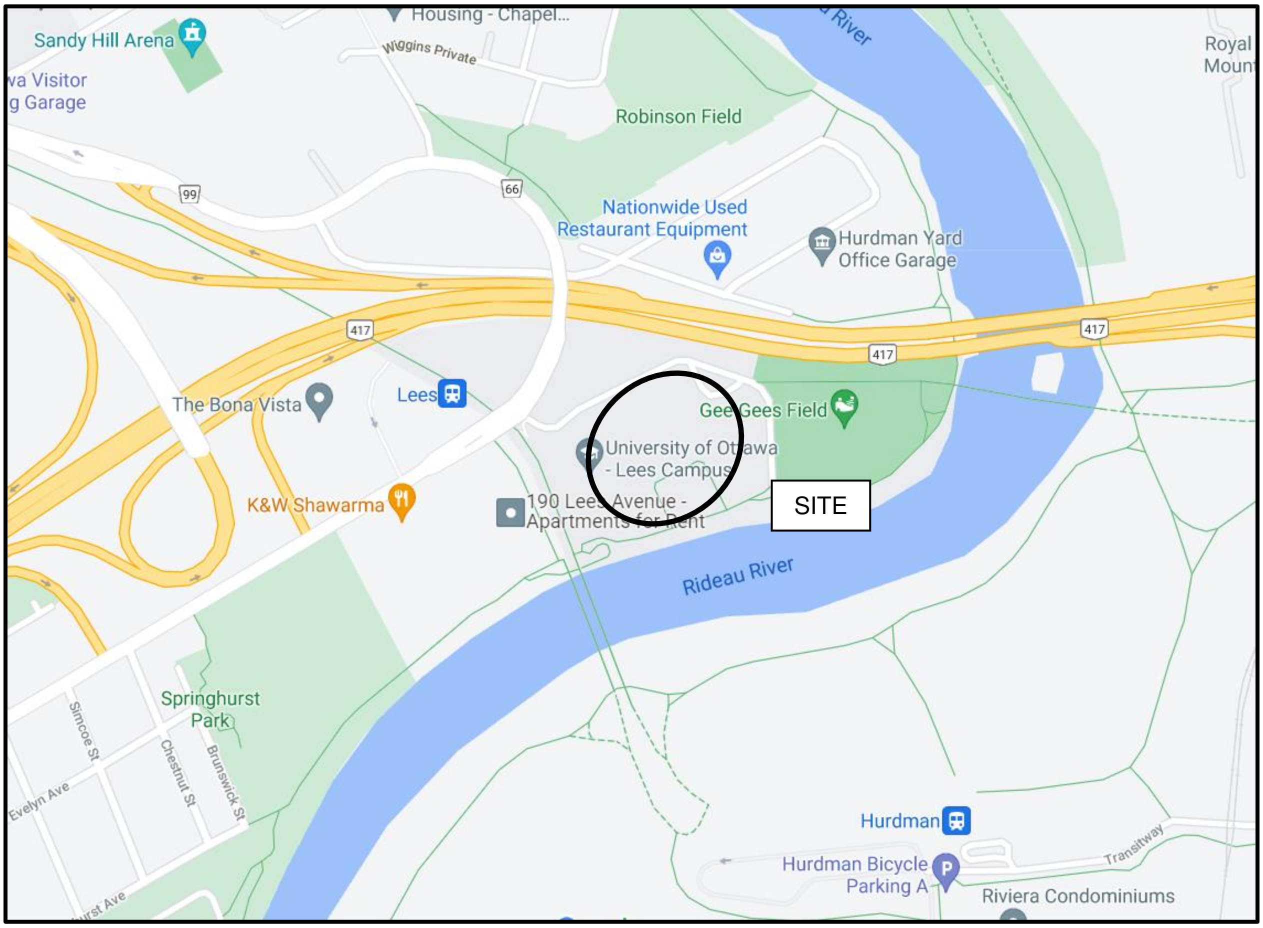
**Client PO: 32181**

Report Date: 21-Jun-2021

Order Date: 15-Jun-2021

**Project Description: PG5656**

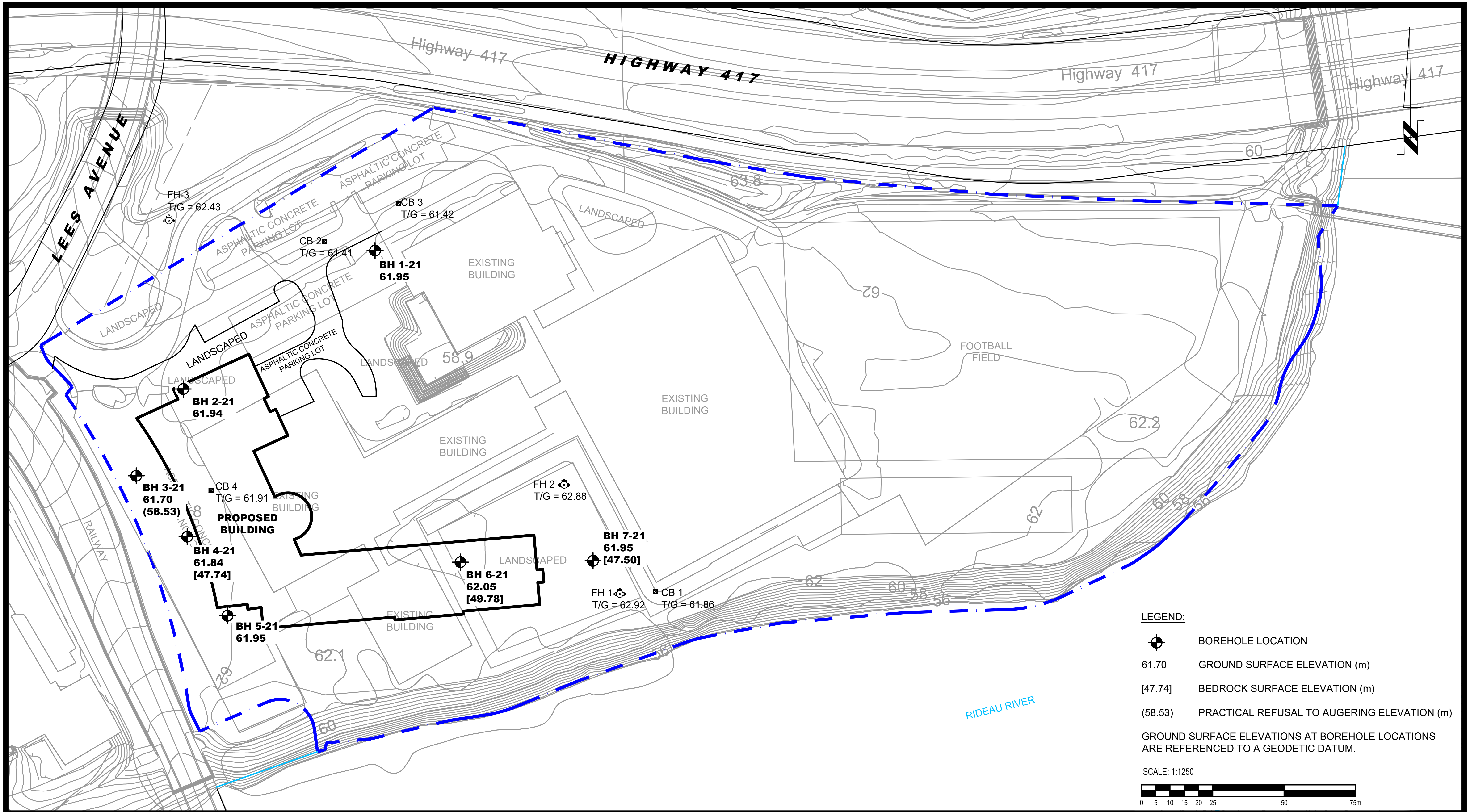
| | | | | | |
|---|---|---|---|---|---|
| | **Client ID:** | BH4-21-SS3 | - | - | - |
| | **Sample Date:** | 14-Jun-21 09:00 | - | - | - |
| | **Sample ID:** | 2125227-01 | - | - | - |
| | **MDL/Units** | Soil | - | - | - |
| **Physical Characteristics** | | | | | |
| % Solids | 0.1 % by Wt. | 82.4 | - | - | - |
| **General Inorganics** | | | | | |
| pH | 0.05 pH Units | 7.23 | - | - | - |
| Resistivity | 0.10 Ohm.m | 2.73 | - | - | - |
| **Anions** | | | | | |
| Chloride | 5 ug/g dry | 1980 | - | - | - |
| Sulphate | 5 ug/g dry | 129 | - | - | - |



# FIGURE 1

## KEY PLAN

**patersongroup**



LEGEND:

| | |
|---|---|
| ⊕ | BOREHOLE LOCATION |
| 61.70 | GROUND SURFACE ELEVATION (m) |
| [47.74] | BEDROCK SURFACE ELEVATION (m) |
| (58.53) | PRACTICAL REFUSAL TO AUGERING ELEVATION (m) |

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:1250

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| NO. | REVISIONS | DATE | INITIAL |
|---|---|---|---|
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PCL c/o U OF OTTAWA HEALTH SCIENCES
GEOTECHNICAL INVESTIGATION
PROPOSED BUILDING
200 LEES AVENUE

OTTAWA, ONTARIO

Title:

# TEST HOLE LOCATION PLAN

Scale: 1:1250
Date: 06/2021
Drawn by: JM
Report No.: PG5656-1
Checked by: JV
Dwg. No.: PG5656-1
Approved by: DJG
Revision No.:

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