

Geotechnical
Engineering

Environmental
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Hydrogeology

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Materials Testing

Building Science

Noise and Vibration
Studies

Geotechnical Investigation

Proposed Mixed-Use Development
265 Catherine Street
Ottawa, Ontario

Prepared For

Brigil

Paterson Group Inc.

Consulting Engineers
154 Colonnade Road
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

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Report PG5933-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Brigil to conduct complete a geotechnical report for the proposed mixed-use development to be located at 265 Catherine Street in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- ❑ Determine the subsoil and groundwater conditions at this site by means of boreholes.
- ❑ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

2.0 Proposed Project

Based on the current concept drawings, it is understood that the proposed development will consist of several mid-rise to hi-rise mixed-use buildings. It is understood that the subject site will be provided two to three levels of underground parking whose footprints are anticipated to occupy the majority of the subject site and provide a common podium to all the proposed structures. At-grade access lanes, parking areas and landscaping areas are also anticipated as part of the proposed development. It is further anticipated that the existing building will be demolished as part of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

Paterson previously conducted a series of field program for the subject site. The most recent investigation was carried out on August 19, 2020. At that time, 3 boreholes (BH 1 through BH 5) were advanced to a maximum depth of 14.7 m below the existing ground surface. Paterson had previously carried out an investigation on August 24 and 25, 2010. At that time, six (6) boreholes were advanced to a maximum sampling depth of 6 m and five (5) of the boreholes were extended to inferred bedrock based on practical refusal to augering. A previous investigation was carried out by others in August of 1971. At that time, five (5) boreholes were advanced to a maximum depth fo 12.5 m.

The borehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the test holes are shown on Drawing PG5933-1 - Test Hole Location Plan included in Appendix 2.

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

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. All samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the auger and split spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted at each borehole 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.

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

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH 2-20. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater monitoring wells were installed in BH1-20, BH 2-20 and BH 3-20 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. A flexible polyethylene standpipe was installed within boreholes from the previous investigation to measure the stabilized groundwater levels subsequent to completion of the sampling program.

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 elevation at each test hole location were surveyed by Paterson with respect to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5933-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was analyzed to determine its concentration of sulphate and chloride along with its resistivity and pH. The laboratory test results are shown in Appendix 1 and are discussed in Section 6.7.

4.0 Observations

4.1 Surface Conditions

Currently, the subject site is occupied by a bus terminal building with associated asphalt covered parking areas and access lanes. The subject site is approximately at grade with surrounding streets.

The site is bordered by Catherine Street to the south, Lyon Street to the west, Arlington Avenue and further by residential dwellings to the north and Kent Street to the east. The existing ground surface across the subject site is relatively flat and at grade with adjacent properties.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of concrete or asphaltic concrete underlain by fill extending to an approximate depth of 0.6 to 2.3 m below the existing ground surface. The fill was generally observed to consist of a compact brown silty sand with crushed stone and occasional brick, metal, and plastic fragments.

A native silty sand layer and/or silty clay deposit was encountered underlying the fill. The silty clay deposit was observed to consist of a very stiff to stiff, brown silty clay, becoming a stiff grey silty clay below an approximate depths ranging between 3.0 to 7.6 m below the existing ground surface.

Underlying the silty clay deposit below approximate depths ranging between 4.4 to 9.7 m, a glacial till layer was encountered. The glacial till deposit was observed to consist of a grey sandy silt, clayey silt or silty clay with gravel, cobbles and boulders.

Practical refusal to augering or the DCPT was encountered at depths ranging from 7.4 to 11.7 m below the existing ground surface.

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and shale of the Verulam Formation and shale of the Billings Formation at depths ranging from 10 to 15 m.

4.3 Groundwater

Groundwater levels measured in the standpipes are summarized in Table 1.

Table 1 - Summary of Groundwater Level Readings				
Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1-20*	68.62	4.60	64.02	September 1, 2020
BH 2-20*	68.46	Dry	-	September 1, 2020
BH 3-20*	68.11	4.26	63.85	September 1, 2020
BH 1	-	3.48	-	September 16, 2010
BH 2	-	5.32	-	September 16, 2010
BH 3	-	5.30	-	September 16, 2010
BH 4	-	N/A	-	September 16, 2010
BH 5	-	4.59	-	September 16, 2010
BH 6	-	2.18	-	September 16, 2010

Note: The ground surface elevations from the current investigation are referenced to a geodetic datum.
* - Borehole fitted with monitoring well.

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 4 to 5 m below ground surface within the silty clay layer. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

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

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. Based on the subsurface conditions encountered in the test holes and the anticipated building loads, it is recommended that foundation support for the proposed high-rise buildings consist of:

- a raft foundation bearing on the undisturbed, stiff silty clay and compact glacial till deposit, or
- a deep foundation, such as end-bearing piles, which extends to the bedrock surface

Due to the presence of the silty clay deposit, a permissible grade raise restriction will be required for the proposed grading.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas and other settlement sensitive structures. Existing construction debris should be entirely removed from within the perimeter of all buildings.

Protection of Subgrade (Raft Foundation)

Where a raft foundation is utilized, it is recommended that a minimum 50 mm thick lean concrete mud slab be placed on the undisturbed, silty clay or glacial till subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the silty clay or glacial till to potential disturbance due to drying.

Compacted Granular Fill Working Platform (Pile Foundation)

Should the proposed high-rise building be supported on a driven pile foundation, the use of heavy equipment would be required to install the piles (i.e. pile driving crane). It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance.

A typical working platform could consist of 0.6 m of OPSS Granular B, Type II crushed stone which is placed and compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in lifts not exceeding 300 mm in thickness.

Once the piles have been driven and cut off, the working platform can be regraded, and soil tracked in, or soil pumping up from the pile installation locations, can be bladed off and the surface can be topped up, if necessary, and re-compacted to act as the substrate for further fill placement for the basement slab.

Fill Placement

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

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Raft Foundation

Based on the anticipated loads, it is recommended that foundation support for the high-rise portion of the proposed building consist of a raft foundation bearing on the undisturbed, stiff silty clay or compact glacial till deposit.

A raft foundation may also be required for the portion of the proposed building beyond the footprint of the high-rise. For 2 levels of underground parking, it is anticipated that the excavation will extend to a depth such that the underside of the raft slab would be placed between geodetic elevations of 61.5 to 62.5 m. The contact pressure provided considers the stress relief associated with the soil removal required for 2 levels of underground parking.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load.

For 2 levels of underground parking, a bearing resistance value at SLS (contact pressure) of **175 kPa** will be considered acceptable for a raft supported on the undisturbed, stiff silty clay or undisturbed glacial till. It should be noted that the weight of the raft slab and everything above must be included when designing with this value. The factored bearing resistance (contact pressure) at ULS can be taken as **260 kPa**. For this case, the modulus of subgrade reaction was calculated to be **5.0 MPa/m** for a contact pressure of **175 kPa**.

The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

Based on the following assumptions for the raft foundation, the proposed building can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

End Bearing Pile Foundation

If the raft slab bearing resistance values are insufficient for the proposed high-rise building, a deep foundation system driven to refusal in the bedrock will be 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 SLS and ULS are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored 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 2 - Pile Foundation Design Data					
Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	940	1130	10	29
245	11	1175	1410	10	35
245	13	1375	1650	10	42

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.

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.

Buildings founded on piles driven to refusal in the bedrock will have negligible post-construction settlement.

Spread Footing Foundations

Foundations may consist of strip footings, up to 4 m wide, and pad footings, up to 8 m wide, placed over an undisturbed, very stiff to stiff silty clay bearing surface using bearing resistance values at Serviceability Limit State (SLS) of **160 kPa** and factored bearing resistance values at Ultimate Limit States (ULS) of **250 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 materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

The bearing resistance values at SLS for conventional style footings will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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 silty clay or glacial till bearing medium 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 or engineered fill of the same or higher capacity as the soil.

Permissible Grade Raise

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **2.0 m** is recommended for grading at the subject site.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D**, however, a site specific shear wave velocity test could be completed to determine if a seismic Site Class C is applicable for foundation design of the proposed building with two underground parking levels. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab

Where a raft slab is utilized, a granular layer of OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

For portions of the proposed building founded on footings or piles, it is recommended that the upper 200 mm of subfloor fill consists 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 at least 98% of its SPMDD.

A sub-slab drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided under the lowest level floor slab. The spacing of the sub-slab drainage pipes can be determined at the time of construction to confirm groundwater infiltration levels, if any. This is 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 .

Where undrained conditions are anticipated (i.e. below the groundwater level), 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/m³)

H = height of the wall (m)

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

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

Seismic Earth Pressures

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

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

$a_c = (1.45 - a_{max}/g)a_{max}$

γ = unit weight of fill of the applicable retained soil (kN/m³)

H = height of the wall (m)

g = gravity, 9.81 m/s²

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

Car only parking areas, heavy truck parking areas and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 3 and 4.

Table 3 - 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 - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil	

Table 4 - Recommended Pavement Structure Access Lanes, Ramp 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 - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

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

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage and Waterproofing

For the proposed underground parking levels, it is expected that the building foundation walls will be placed in close proximity to the site boundaries. Therefore, it is recommended that the foundation walls be blind poured against a drainage system and waterproofing system fastened to the shoring system.

Waterproofing of the foundation walls is recommended and the membrane is to be installed from 300 mm above the proposed P1 level to the bottom of foundation. It is further recommended to extend the membrane a minimum of 600 mm horizontally below the exterior footing as per Figure 2 - Water Suppression System in Appendix 2.

It is also recommended that a composite drainage system, such as Delta Drain 6000 or equivalent, be installed between the waterproofing membrane and the foundation wall, and extend from the exterior finished grade to the founding elevation (underside of footing or raft slab). The purpose of the composite drainage system is to direct any water infiltration resulting from a breach of the waterproofing membrane to the building sump pit. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the perimeter footing or raft slab interface to allow the infiltration of water to flow to an interior perimeter underfloor drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Foundation Raft Slab Construction Joints

It is expected that the raft slab, where utilized, will be poured in sections. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration below the lowest level floor slabs. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centres. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where space is available for conventional wall construction, 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 A crushed stone, should be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover, or a minimum of 0.6 m of soil cover in conjunction with adequate foundation insulation, should be provided.

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

The foundations for the underground parking levels are expected to have sufficient frost protection due to the founding depth. However, it has been our experience that insufficient soil cover is typically provided to entrance ramps to underground parking garages. Paterson requests permission to review design drawings prior to construction to ensure proper frost protection is provided for these areas.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled. Given the proximity of the underground parking levels to the property lines, it is expected that a temporary shoring will be required to support the excavation for this proposed development.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring will be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer.

Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated with the following parameters.

Table 5 - 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	21
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 and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

The pipe bedding for sewer and water pipes placed on a relatively dry, undisturbed subgrade surface should consist of at least 150 mm of OPSS Granular A material. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

Generally, it should be possible to re-use the moist (not wet) silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay material will be difficult to re-use, as the high water contents make compacting impractical without an extensive drying period.

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

6.5 Groundwater Control

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. The rate of flow of groundwater into the excavation through the overburden should be low to moderate for the conditions expected at this site. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

Groundwater Control for Building Construction

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 on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater which breaches the building's perimeter groundwater infiltration control system will be directed to the sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that long-term groundwater flow will be very low to negligible (ie.- less than 30,000 L/day).

Impacts on Neighbouring Structures

Based on our observations, the groundwater level is anticipated at a 4 to 5 m depth and within the silty clay layer. Therefore, a local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. Since the proposed development will be founded below the long term groundwater level, a groundwater infiltration control system has been recommended to lessen the effects of water infiltration. Any long term dewatering of the site will be minimal and should have no adverse effects to the surrounding buildings or structures. The short term dewatering during the excavation program will be managed by the excavation contractor, as discussed above.

6.6 Winter Construction

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

The subsoil conditions at this site mostly 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, 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.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a non-aggressive to very aggressive 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 grading plan from a geotechnical perspective.
- Review the Contractor's design of the temporary shoring system.
- Review of waterproofing details for elevator shaft and building sump pits.
- Review and inspection of the foundation waterproofing system and all foundation drainage 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.
- 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 our present understanding of the project. We request permission to review our 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, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine 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 Brigil 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.



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



David J. Gilbert, P.Eng.

Report Distribution

- Brigil
- Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

SOIL PROFILE AND TEST DATA SHEETS BY OTHERS

ANALYTICAL TESTING RESULTS

DATUM Geodetic

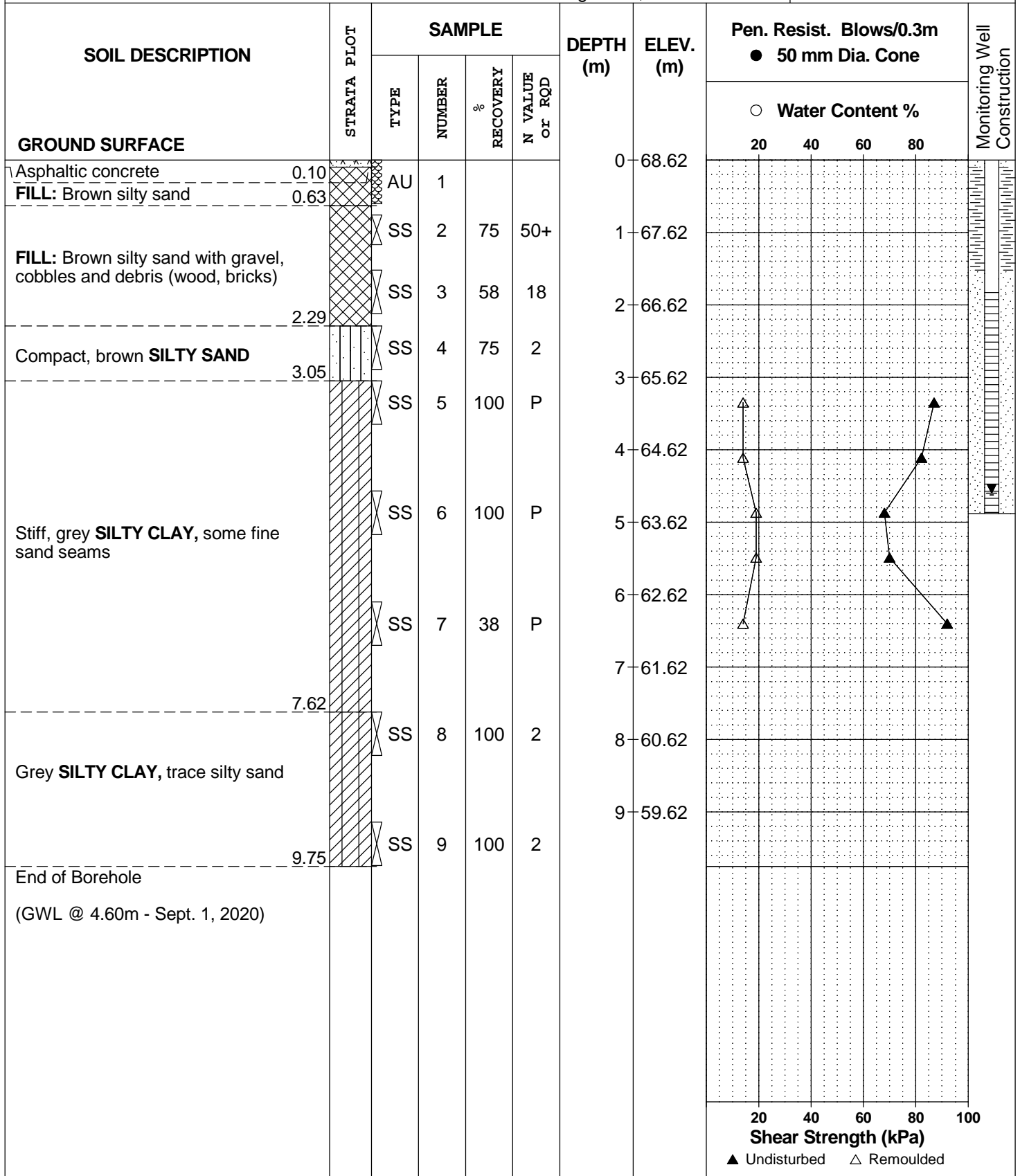
REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE August 19, 2020

FILE NO. **PG5498**

HOLE NO. **BH 1-20**



DATUM Geodetic

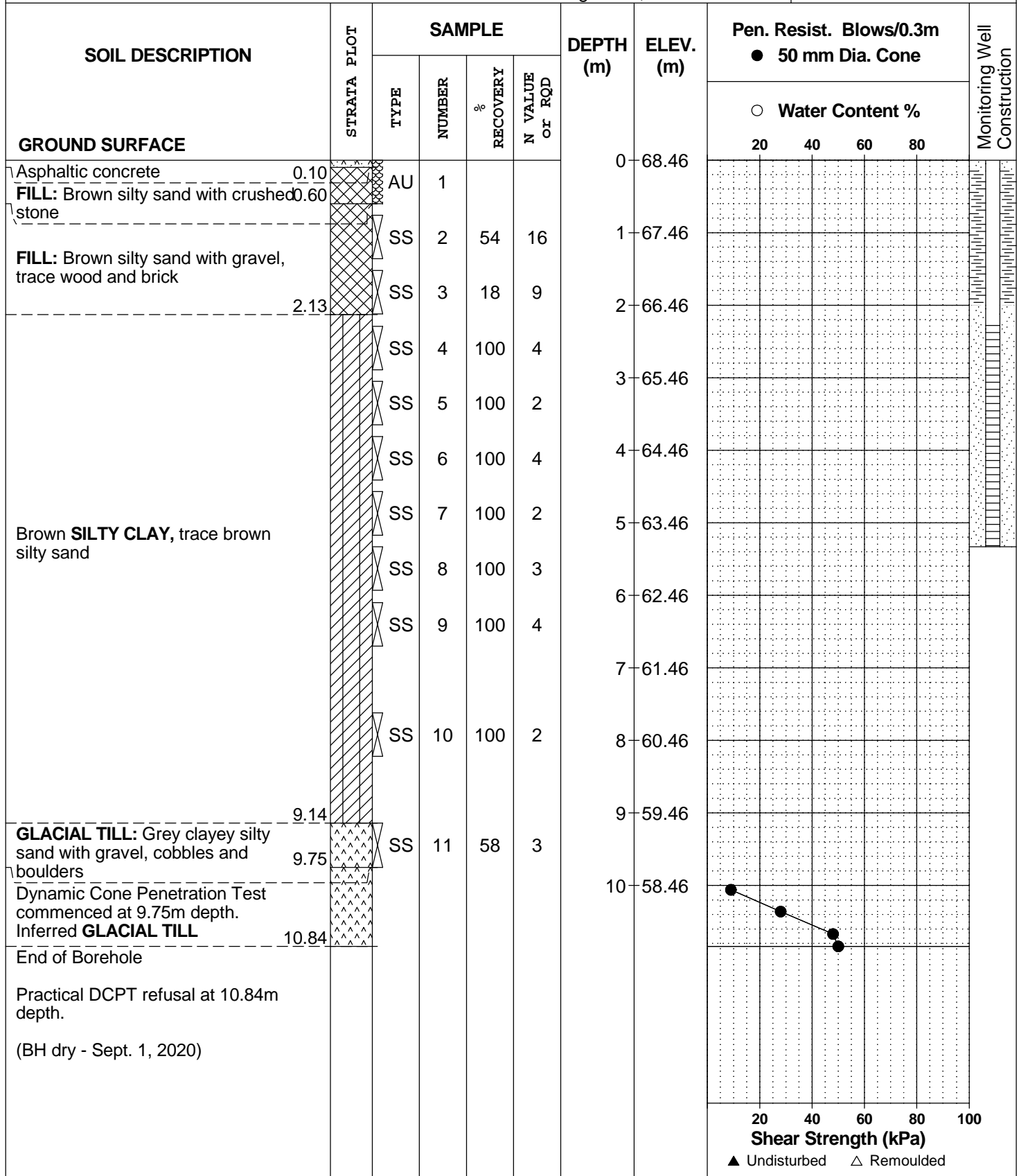
FILE NO. **PG5498**

REMARKS

HOLE NO. **BH 2-20**

BORINGS BY CME-55 Low Clearance Drill

DATE August 19, 2020



DATUM Geodetic

FILE NO. **PG5498**

REMARKS

HOLE NO. **BH 3-20**

BORINGS BY CME-55 Low Clearance Drill

DATE August 19, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Monitoring Well Construction	
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD			○ Water Content %					
								20	40	60	80		
GROUND SURFACE													
Asphaltic concrete	0.10					0	68.11						
FILL: Brown silty sand with silty clay and crushed stone	0.60	AU	1										
Loose to compact, brown SILTY SAND , some organics		SS	2	38	9	1	67.11						
		SS	3	67	13	2	66.11						
	2.13												
		SS	4	100	2	3	65.11						
Stiff, grey SILTY CLAY with sandy silt		SS	5	100	2	4	64.11						
	4.42												
GLACIAL TILL: Compact, grey sandy silt with some clay, gravel and cobbles		SS	7	42	11	5	63.11						
	5.33												
		SS	8	62	4	6	62.11						
GLACIAL TILL: Grey clayey silty sand with gravel, cobbles and boulders		SS	8	46	7	7	61.11						
	7.49												
End of Borehole													
Practical refusal to augering at 7.49m depth. (GWL @ 4.26m - August 28, 2020)													

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

The standard terminology to describe the 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)
D _{xx}	-	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
D ₁₀	-	Grain size at which 10% of the soil is finer (effective grain size)
D ₆₀	-	Grain size at which 60% of the soil is finer
C _c	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
C _u	-	Uniformity coefficient = D_{60} / D_{10}

C_c and C_u are used to assess the grading of sands and gravels:

Well-graded gravels have: $1 < C_c < 3$ and $C_u > 4$

Well-graded sands have: $1 < C_c < 3$ and $C_u > 6$

Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.

C_c and C_u 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
C _{cr}	-	Recompression index (in effect at pressures below p' _c)
C _c	-	Compression index (in effect at pressures above p' _c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
W _o	-	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.
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SYMBOLS AND TERMS (continued)

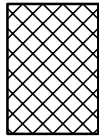
STRATA PLOT



Topsoil



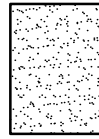
Asphalt



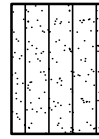
Fill



Peat



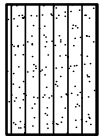
Sand



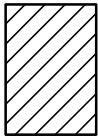
Silty Sand



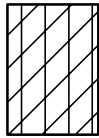
Silt



Sandy Silt



Clay



Silty Clay



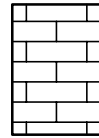
Clayey Silty Sand



Glacial Till



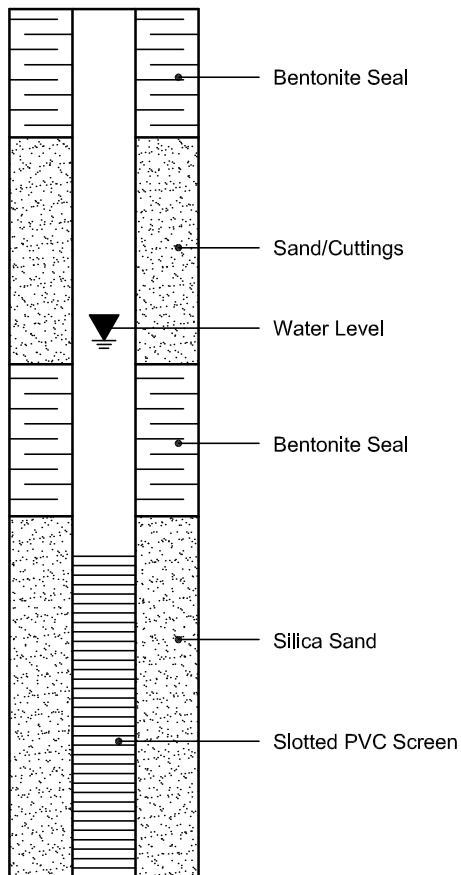
Shale



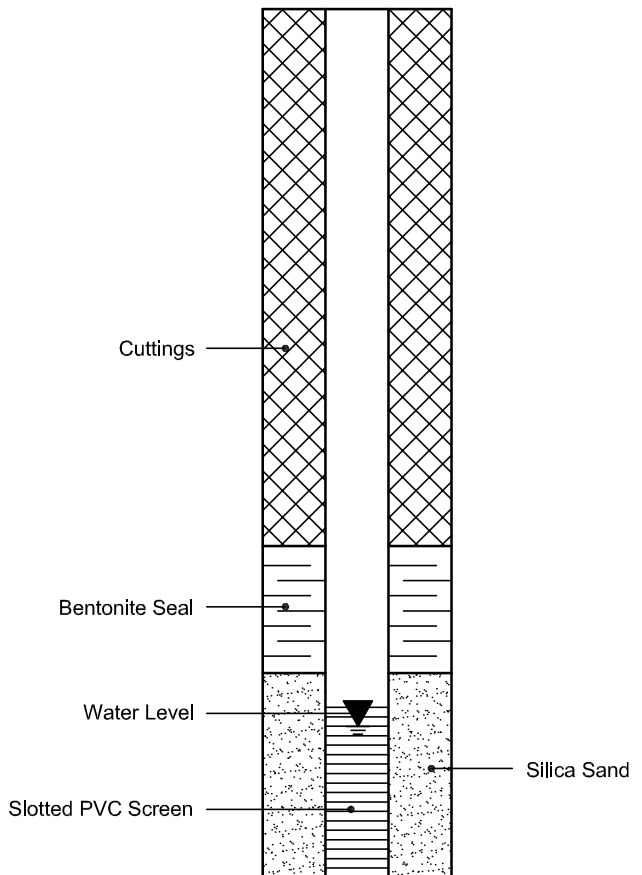
Bedrock

MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



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OTTAWA CANADA

SOIL PROFILE & TEST SUMMARIES

PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

KENT & LYON

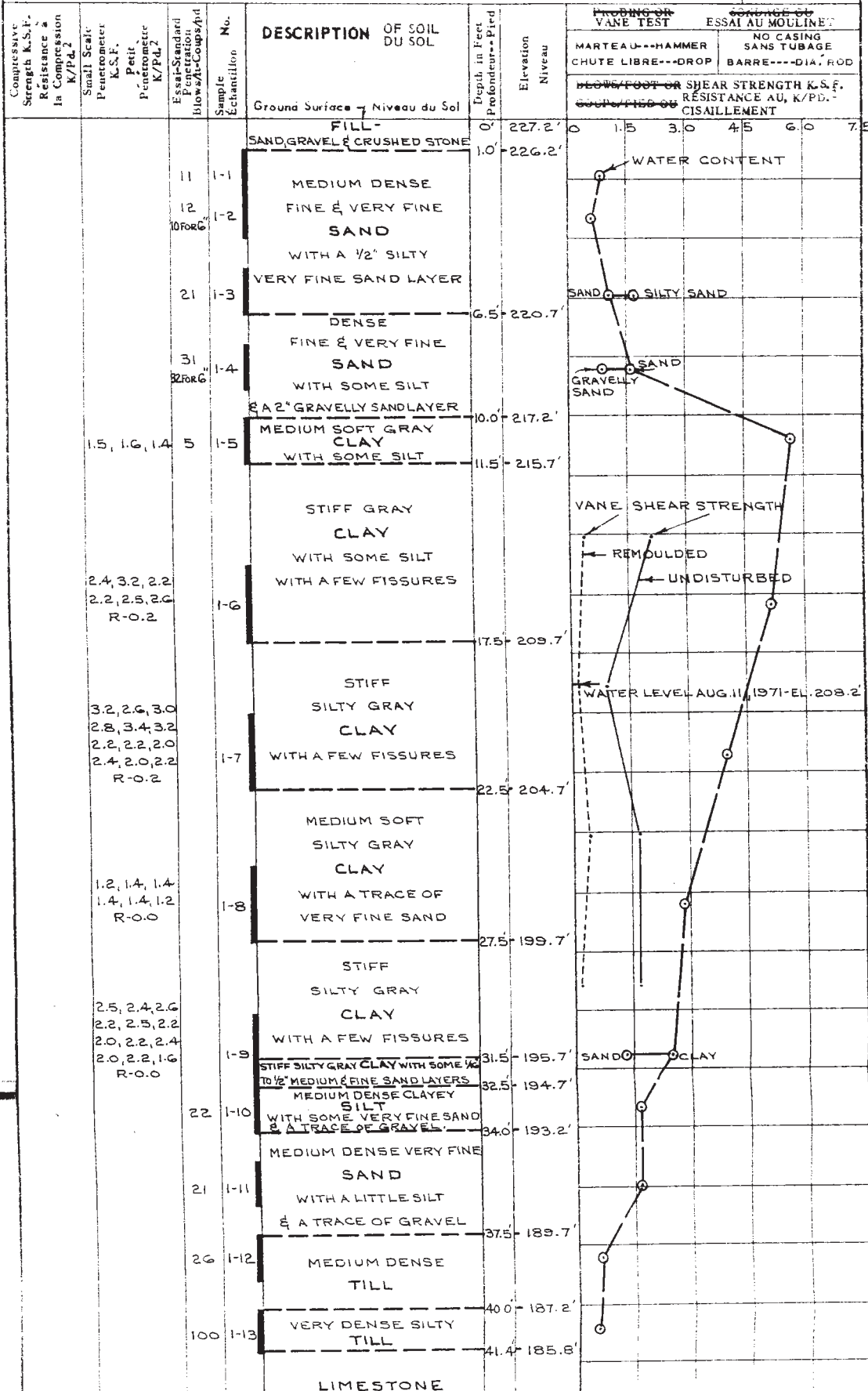
ELEVATION OF GROUND SURFACE (ZERO DEPTH) 227.2'

DATE AUG. 6, 1971

HOLE FORAGE No. 1

NIVEAU DU SOL (PROFONDEUR ZÉRO)

NOTES: B.M. (ELEV. 231.49') GEODETIC, SOUTHEAST CORNER OF KENT & MCLEOD



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PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

KENT & LYON

ELEVATION OF GROUND SURFACE (ZERO DEPTH)
NIVEAU DU SOL (PROFONDEUR ZERO)

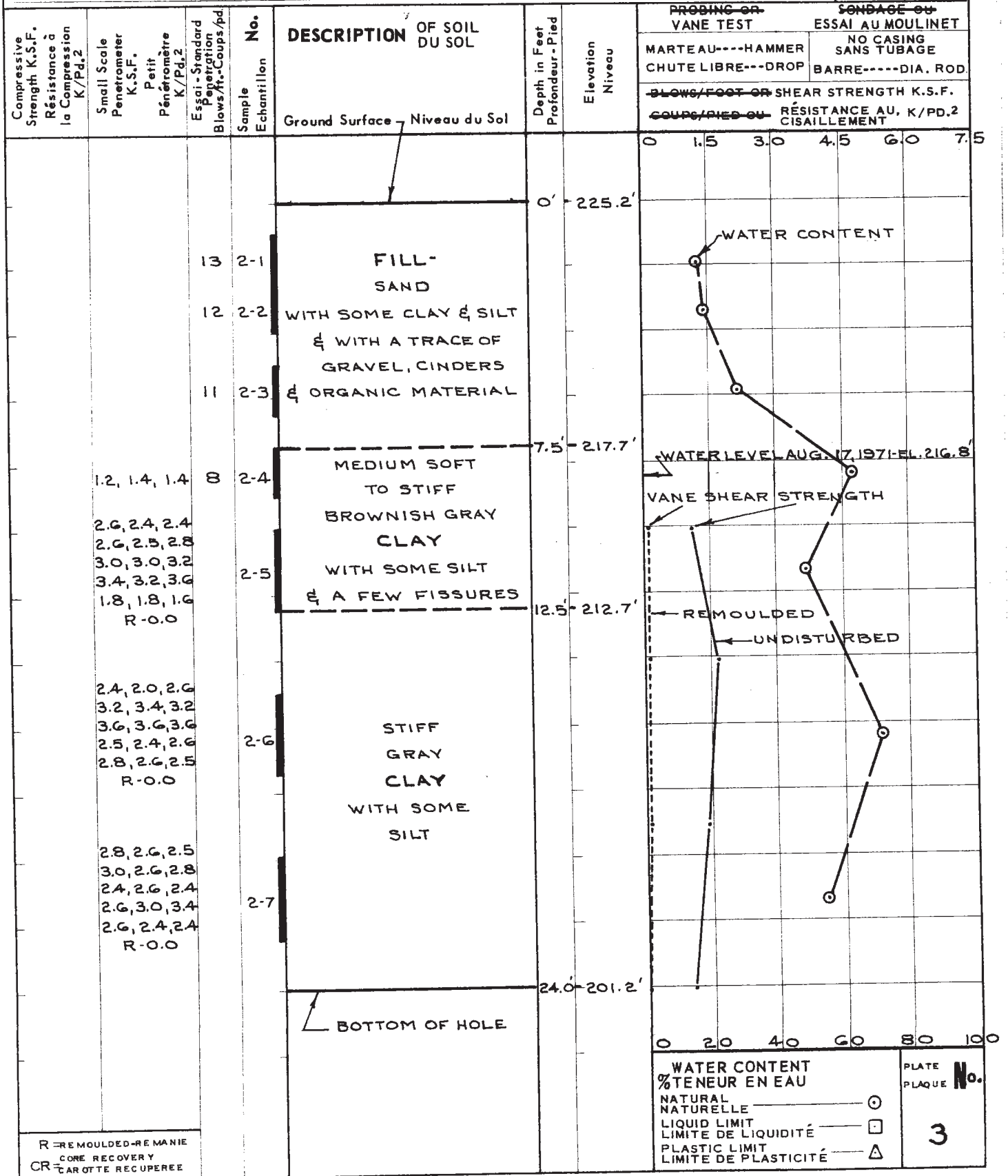
225.2'

DATE AUG. 9, 1971

HOLE FORAGE No.

2

NOTES SEE PLATE No. 2



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SOIL PROFILE & TEST SUMMARIES

PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

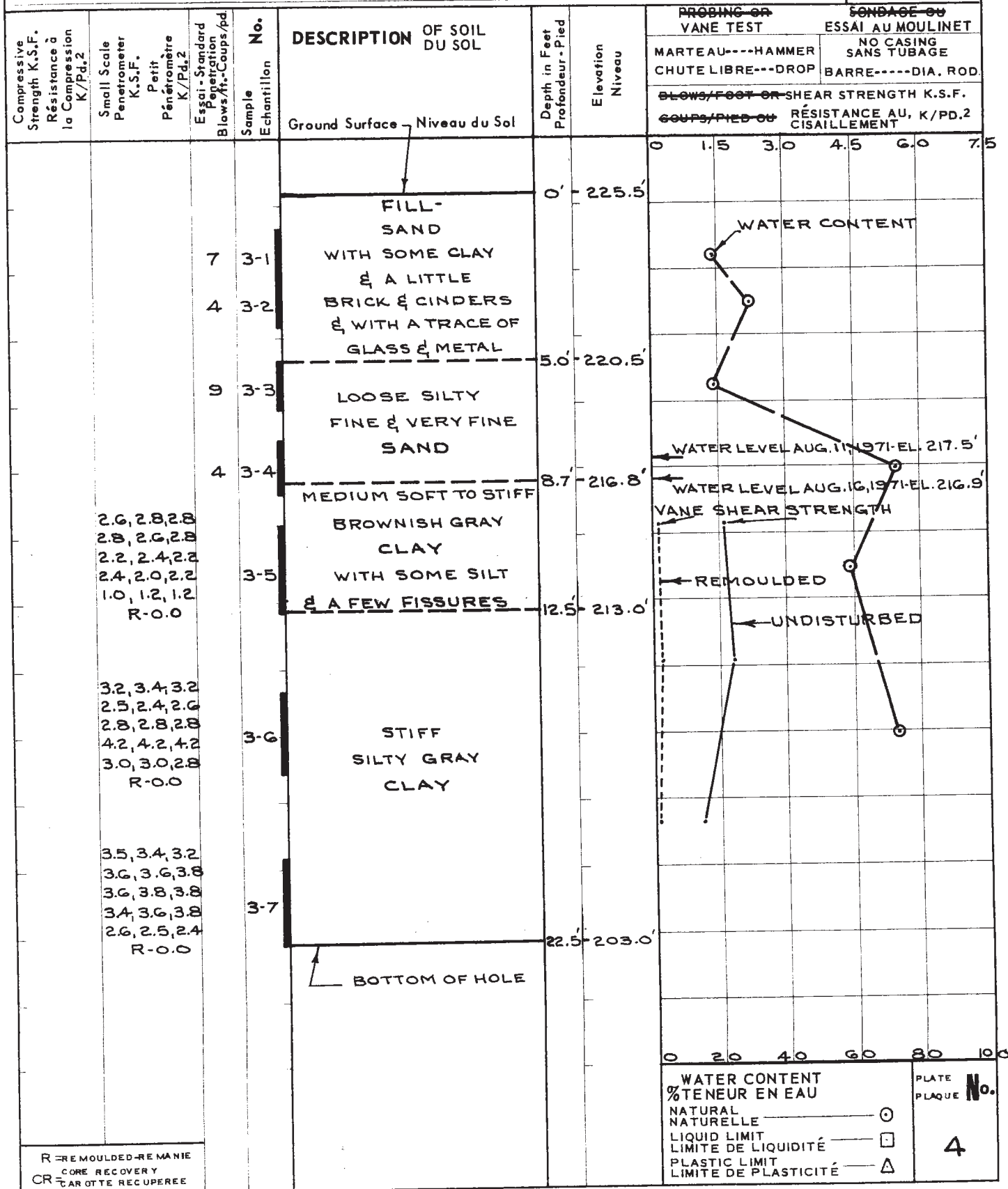
KENT & LYON

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 225.5'
NIVEAU DU SOL (PROFONDEUR ZERO)

DATE AUG. 9, 1971

HOLE FORAGE No. 3

NOTES SEE PLATE No. 2



R = REMOULDED-RE MANIE
CORE RECOVERY
CR = CAR OTTE RECUPEREE

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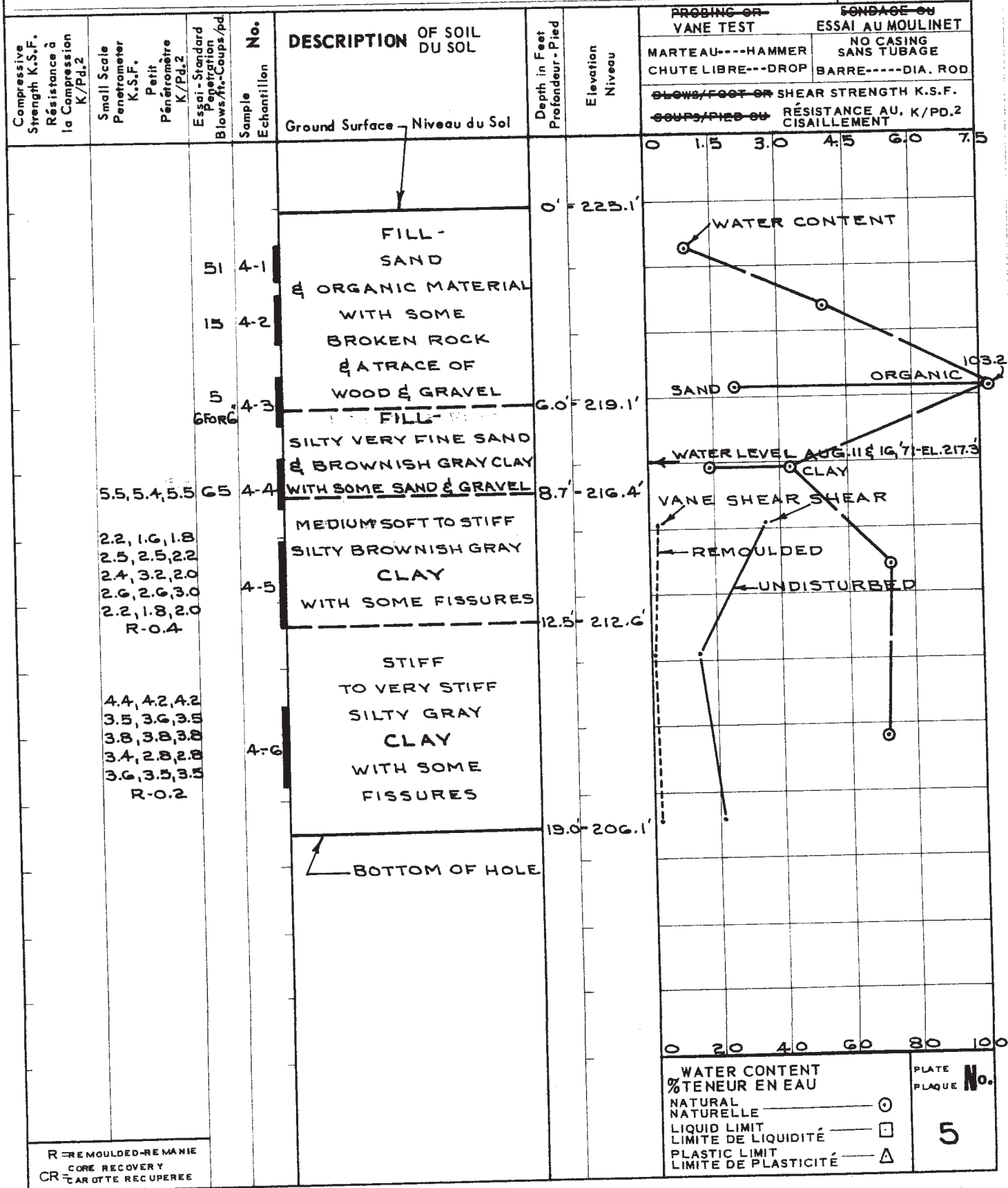
SOIL PROFILE & TEST SUMMARIES
 PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

KENT & LYON

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 225.1' DATE AUG. 9, 1971
 NIVEAU DU SOL (PROFONDEUR ZERO)

HOLE FORAGE No. 4

NOTES SEE PLATE No. 2



R = REMOULDED - RE MANIE
 CORE RECOVERY
 CR = CAROTTE RECUPEREE

WATER CONTENT
 % TENEUR EN EAU
 NATURAL / NATURELLE ○
 LIQUID LIMIT / LIMITE DE LIQUIDITE □
 PLASTIC LIMIT / LIMITE DE PLASTICITE △

PLATE PLAQUE No. 5

Certificate of Analysis

Report Date: 26-Aug-2020

Client: Paterson Group Consulting Engineers

Order Date: 20-Aug-2020

Client PO: 30690

Project Description: PE2703

Client ID:	BH3-20 SS4	-	-	-
Sample Date:	19-Aug-20 09:00	-	-	-
Sample ID:	2034480-01	-	-	-
MDL/Units	Soil	-	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	59.6	-	-	-
----------	--------------	------	---	---	---

General Inorganics

pH	0.05 pH Units	7.40	-	-	-
Resistivity	0.10 Ohm.m	3.33	-	-	-

Anions

Chloride	5 ug/g dry	1780	-	-	-
Sulphate	5 ug/g dry	398	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

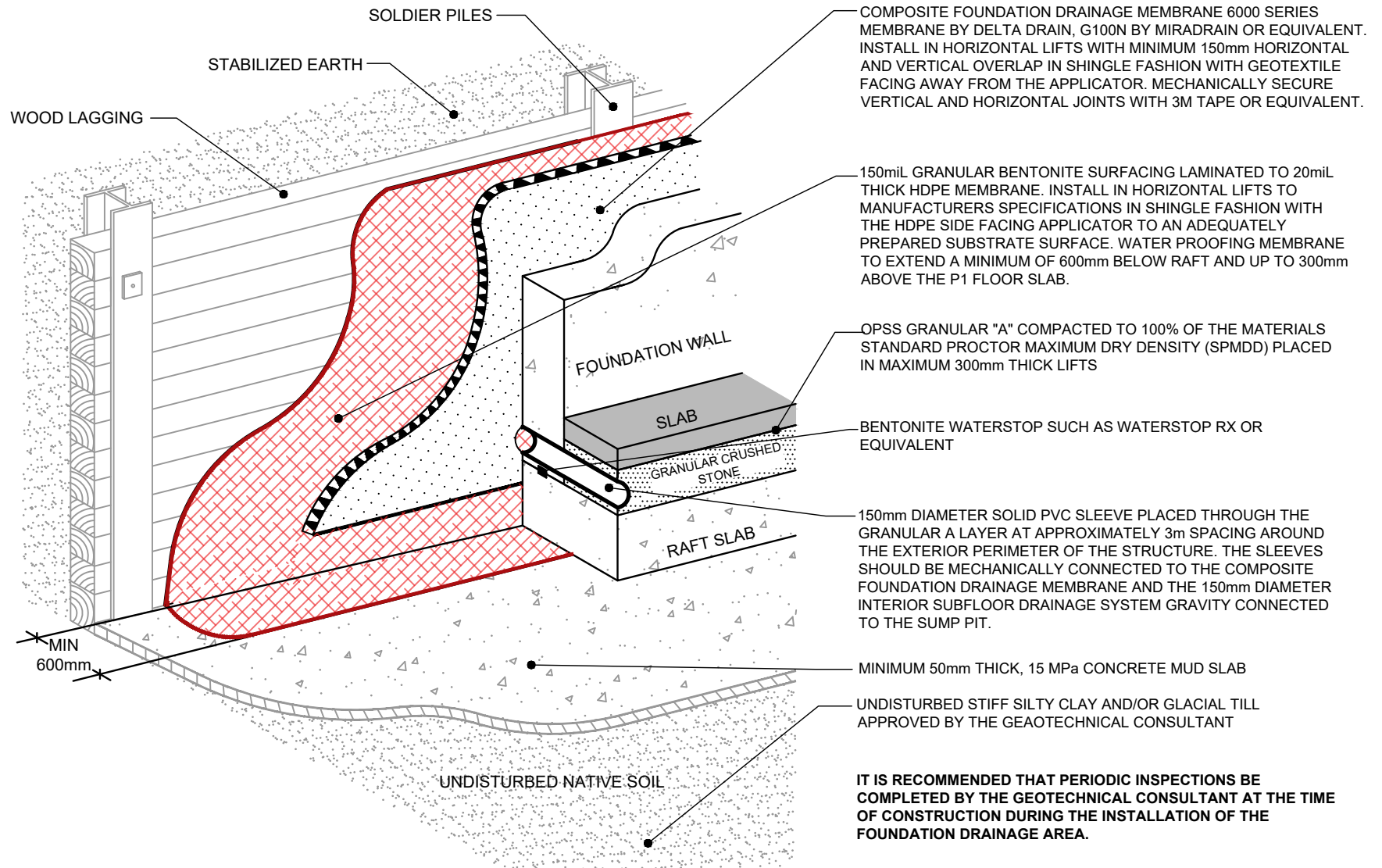
FIGURE 2 - WATER SUPPRESSION SYSTEM

DRAWING PG5933-1 - TEST HOLE LOCATION PLAN



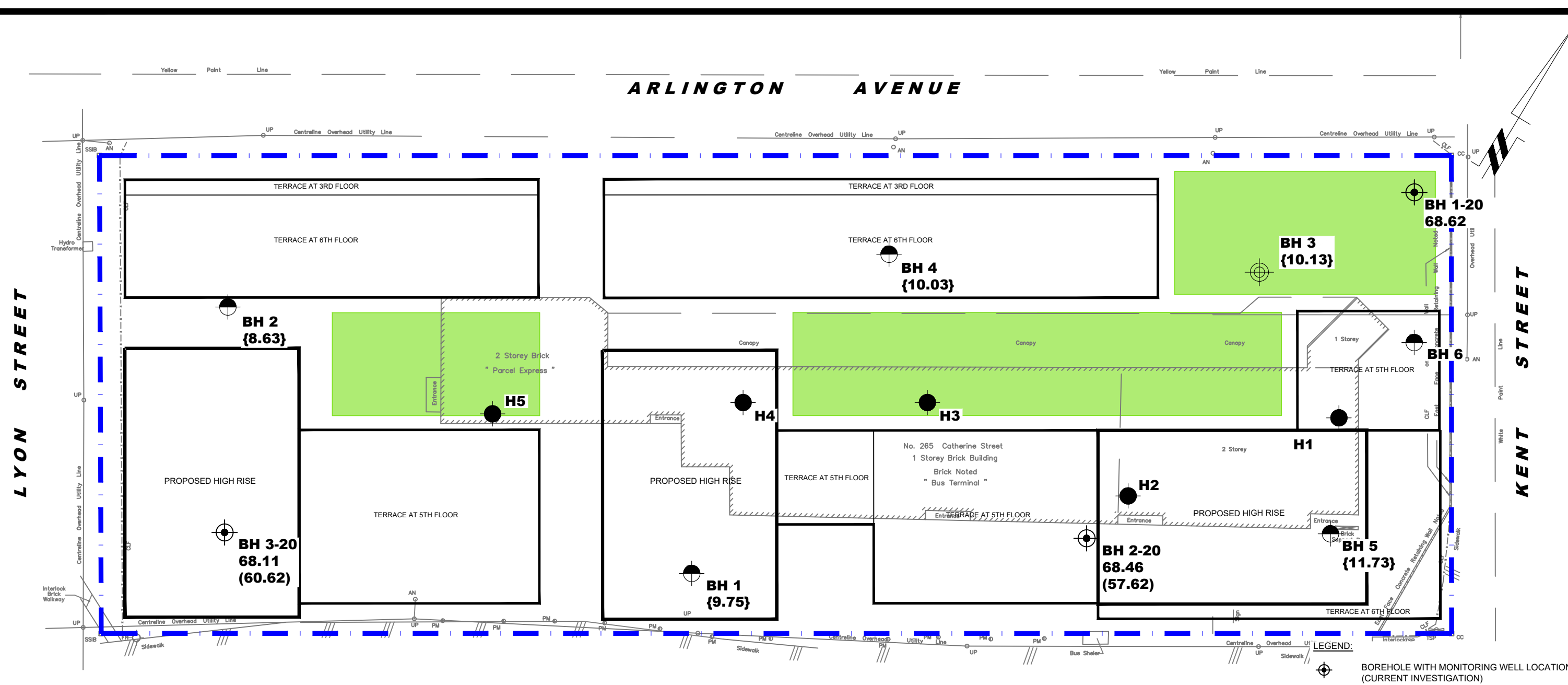
FIGURE 1

KEY PLAN

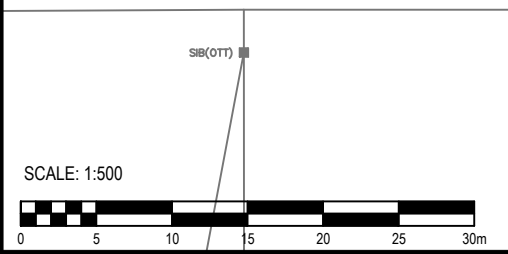


IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE FOUNDATION DRAINAGE AREA.

<p>patersongroup consulting engineers</p> <p>154 Colonnade Road South Ottawa, Ontario K2E 7J5 Tel: (613) 226-7381 Fax: (613) 226-6344 www.patersongroup.ca</p>	<p>BRIGIL CONSTRUCTION PROPOSED HI-RISE BUILDING 265 CATHERINE STREET</p>		Scale: NTS	Date: 08/2021
	<p>OTTAWA, ONTARIO</p>		Drawn by: NFRV	Report No.: PG5933
	<p>WATER SUPPRESSION SYSTEM</p>		Checked by: DP	<p>FIG. 2</p>
			Approved by: DJG	



- LEGEND:**
- BOREHOLE WITH MONITORING WELL LOCATION (CURRENT INVESTIGATION)
 - BOREHOLE LOCATION (PREVIOUS INVESTIGATION PG2174-1, SEPTEMBER, 2010)
 - BOREHOLE WITH MONITORING WELL LOCATION (PREVIOUS INVESTIGATION, PG2174-1, SEPTEMBER, 2010)
 - BOREHOLE LOCATION BY OTHERS FROM PREVIOUS INVESTIGATION
 - 68.46 GROUND SURFACE ELEVATION (m)
 - {11.73} PRACTICAL REFUSAL DEPTH (m)
 - (57.62) PRACTICAL REFUSAL TO DCPT / AUGERING ELEVATION (m)
- GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.



patersongroup
 consulting engineers

154 Colonnade Road South
 Ottawa, Ontario K2E 7J5
 Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

BRIGIL CONSTRUCTION

PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED HI-RISE BUILDING - 265 CATHERINE STREET

OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Scale:	1:500	Date:	07/2021
Drawn by:	RCG	Report No.:	PG5933-1
Checked by:	BN	Dwg. No.:	PG5933-1
Approved by:	JV	Revision No.:	

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