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Geotechnical Engineering

Environmental Engineering

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Materials Testing

Building Science

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Geotechnical Investigation

Proposed Multi-Storey Building
2425 Bank Street
Ottawa, Ontario

Prepared For

Waterford Ottawa Seniors Residence

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Report PG5163-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Waterford Ottawa Seniors Residence to conduct a geotechnical investigation for the proposed multi-storey building to be located at 2425 Bank 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 available drawings, it is understood that the proposed development will include a multi-storey building addition, constructed immediately to the east of the existing building. The multi-storey building addition will have 3 levels of underground parking which will extend beyond the limits of the multi-storey building addition to the north and east.

Four single-storey additions, 3 with a slab-on-grade and 1 with a basement level, are also proposed on the north, east, southeast, and south portions of the existing building.

New asphalt paved access lanes and parking areas with landscaped margins are also planned in the vicinity of the proposed multi-storey addition.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the geotechnical investigation was carried out on November 22 and 23, 2019. At that time, 3 boreholes were advanced to a maximum depth of 15 m below the existing ground surface. Previous geotechnical investigations were also conducted at the site by Paterson consisting of 8 boreholes in September 1999 to a maximum depth of 9.8 m, and 3 test pits in November 1999 to a maximum depth of 2.7 m. The test hole 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 PG5163-1 - 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, sampling and testing the overburden.

The test pit procedure consisted of excavating to the required depths at the selected locations and sampling the overburden. The test pits were backfilled with the excavated soil upon completion.

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. Soil samples were also recovered from the sidewalls of the test pits (G). All soil samples were visually inspected and classified on site. The soil samples were placed in sealed plastic bags and transported to our laboratory for further examination and classification. The depths at which the auger, split spoon, and grab samples were recovered from the test holes are shown as AU, SS, and G, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

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

The thickness of the silty clay layer was evaluated during the course of the investigations by a dynamic cone penetration test (DCPT) at BH 1-19 and BH 3-19, and BH 1 through BH 4. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its 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.

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. The ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), consisting of the top of grate of the existing catch basin located in the parking lot of the subject site with a geodetic elevation of 89.65 m. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5163-1 - 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.

Two Shelby tube samples from the previous geotechnical investigation in September 1999 had been submitted for unidimensional consolidation testing. The results of the consolidation testing are presented on the Consolidation Test result sheets presented in Appendix 1.

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

The subject site is currently occupied by a seniors residence building which varies from 1 to 6 storeys in height. The remainder of the site is generally occupied by asphalt paved access lanes and parking areas with landscaped margins.

The site is bordered by Bank Street to the west, Hunt Club Road to the south, commercial and residential properties to the east, Southgate Road to the northeast, and a commercial property to the northwest. The existing ground surface across the site is relatively level at approximate geodetic elevation 89 to 90 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of asphalt underlain by fill extending to an approximate depth of 1.2 to 3.1 m below the existing ground surface. The fill was generally observed to consist of a loose, brown sand to silty sand with occasional gravel and brick.

A silty clay deposit to clayey silt deposit was encountered underlying the fill. This deposit was observed to consist of a very stiff to stiff, brown silty clay, becoming a firm to soft, grey silty clay to clayey silt at approximate depths of 3 to 4 m below the existing ground surface.

Practical refusal of the DCPTs were encountered at depths ranging from 16.8 m at BH 4 to 25.2 m at BH 3-19.

Bedrock

Based on available geological mapping, the bedrock at the subject site consists of shale of the Carlsbad or Billings formation with a drift thickness of 25 to 50 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	-	1.50	-	October 1, 1999
BH 2	-	1.20	-	October 1, 1999
BH 3	-	2.10	-	October 1, 1999
BH 4	-	Blocked	-	October 1, 1999
BH 1-19	89.37	3.10	86.27	December 3, 2019
BH 2-19	89.78	4.19	85.59	December 3, 2019
BH 3-19	90.12	Inaccessible	-	December 3, 2019

Note:
- The ground surface elevations at the borehole locations are referenced to a TBM, consisting of the top of grate of the existing catch basin located in the parking lot of the site with geodetic elevation = 89.65 m.

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 3 to 4 m below ground surface. 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 multi-storey building addition consist of:

☐ a raft foundation bearing on the undisturbed, firm silty clay deposit, or

☐ a deep foundation, such as end-bearing piles, which extend to the bedrock surface.

The piles could also be utilized to provide foundation uplift resistance. However, should the pile uplift resistance capacities provided in Section 5.3 be insufficient for the foundation uplift loads, rock anchors should be utilized. These are discussed further in Section 5.4.

Foundation support for the single-storey additions to the existing building, and the underground parking levels beyond the footprint of the proposed multi-storey building addition, are recommended to consist of conventional spread footings bearing on the undisturbed, firm silty clay.

It is further recommended that a construction joint be provided to allow for differential settlement between the proposed single-storey additions and the existing structures, which are either supported on piles or are supported on footings which have mostly completed settlement under the current building loads. If differential settlements of up to 25 mm are not tolerable between the proposed single-storey additions and the existing structures, the proposed additions should also be supported on piles.

Due to the presence of the silty clay deposit, a permissible grade raise restriction will be applied for the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below final grade.

Protection of Subgrade (Raft Foundation)

Should a raft foundation be utilized for the proposed multi-storey building addition, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed silty clay subgrade shortly after the completion of the excavation. The main purpose of the mudslab 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 to potential disturbance due to drying.

Compacted Granular Fill Working Platform (Pile Foundation)

Should the proposed multi-storey building addition 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 recompacted 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

Spread Footings for Single-Storey Additions and Underground Parking Levels

Foundations for the single-storey additions to the existing building, and the underground parking levels beyond the footprint of the proposed multi-storey building addition, are recommended to consist of strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed over an undisturbed, firm silty clay bearing surface at bearing resistance values at Serviceability Limit State (SLS) of **100 kPa** and factored bearing resistance values at Ultimate Limit States (ULS) of **180 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.

Footings designed using the bearing resistance value at SLS provided herein 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 firm silty clay bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V (or flatter) passes only through in situ soil or engineered fill.

Raft Foundation for Multi-Storey Building Addition

The proposed multi-storey building addition may be supported on a raft foundation where the contact pressure is within the values provided below. For 3 levels of underground parking, it is expected that the excavation will extend between 10 to 12 m below existing ground surface.

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. The contact pressure provided considers the stress relief associated with the soil removal required for 3 levels of underground parking.

For 3 levels of underground parking, a bearing resistance value at SLS (contact pressure) of **200 kPa** will be considered acceptable for a raft supported on the undisturbed, firm silty clay. The factored bearing resistance (contact pressure) at ULS can be taken as **350 kPa**. For this case, the modulus of subgrade reaction was calculated to be **6 MPa/m** for a contact pressure of **200 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 multi-storey building addition can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

Pile Foundation for Multi-Storey Building Addition

A deep foundation system driven to refusal in the bedrock is also recommended for foundation support of the proposed multi-storey building addition. 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	925	1110	6	27
245	11	1050	1260	6	31
245	13	1200	1440	6	35

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.

Foundation 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 native silty clay and for typical backfill materials compacted to 95% of MPMDD in 300 mm lift thicknesses are provided in Table 3, below, along with the associated earth pressure coefficients for horizontal resistance calculated for footings under lateral loads or deadman anchors. 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 = factored passive earth pressure coefficient of the applicable retained soil, 1.5
γ = 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 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.

Foundation Uplift Resistance

Uplift forces on the proposed foundations can be resisted using the dead weight of the concrete foundations, the weight of the materials overlying the foundations, and the submerged weight of the piles, where utilized. Unit weights of materials are provided in Table 3.

For soil above the groundwater level, calculate using the "drained" unit weight and below groundwater level use the "effective" unit weight. Backfilled excavations in low permeability soils can be expected to fill with water and the use of the effective unit weights would be prudent if drainage of the anchor footings is not provided.

As noted above, piles would generally be located below the groundwater level, so the submerged, or effective, weight of the pile will be available to contribute to the uplift resistance, if required. Considering that this is a reliable uplift resistance, and is really counteracting a dead load, it is our opinion that a resistance factor of 0.9 is applicable for the ULS weight component.

Should the pile uplift resistance capacities be insufficient for the foundation uplift loads, rock anchors should be utilized. This is discussed further in Section 5.4.

A sieve analysis and standard Proctor test should be completed on each of the fill materials proposed to obtain an accurate soil density to be expected, so the applicable unit weights can be estimated.

Table 3 - Geotechnical Parameters for Uplift and Lateral Resistance Design

Material Description	Unit Weight (kN/m^3)		Internal Friction Angle (°) φ'	Friction Factor, tan δ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_A	At-Rest K_O	Passive K_P
OPSS Granular A (Crushed Stone)	22.0	13.7	38	0.60	0.22	0.36	8.8
OPSS Granular B, Type II (Well-Graded Sand-Gravel)	21.5	13.4	36	0.55	0.26	0.41	7.5
In Situ Silty Clay	17.0	10.0	33	0.40	0.30	0.45	3.4

Notes:
❏ Properties for fill materials are for condition of 98% of standard Proctor maximum dry density.
❏ The earth pressure coefficients provided are for horizontal backfill profile.
❏ Passive pressure coefficients incorporate wall friction of 0.5 φ'.

Permissible Grade Raise

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **0.6 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 Rock Anchor Design

Should the foundation uplift resistance capacities, provided above, be insufficient for the foundation uplift loads, rock anchors should be utilized.

Overview of Anchor Features

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 a 60 to 90 degree pullout of rock cone with the apex of the cone near the middle of the bonded length of the anchor. 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 individual anchor. A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout does not flow from one hole to an adjacent empty one.
The anchor be provided with a bonded length at the base of the anchor which will provide the anchor capacity, as well an unbonded length between the rock surface and the top of the bonded length.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long term performance of the foundation of the proposed buildings, the rock anchors, where required for this project, are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of shale ranges between about 50 and 80 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 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 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.821 and 0.00293**, 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	40 MPa
Rock Mass Rating (RMR) - Good quality Shale Hoek and Brown parameters	65 m=0.821 and s=0.00293
Unconfined compressive strength - Shale	50 MPa
Unit weight - Submerged Bedrock	15.2 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 5. The factored tensile resistance values given in Table 5are based on a single anchor with no group influence effects. A detailed analysis of the anchorage system, including potential group influence effects, could be provided once the details of the loading for the proposed buildings are determined.

Table 5 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2.0	0.8	2.8	450
	2.6	1.0	3.6	600
	3.2	1.2	4.4	750
	4.5	2.0	6.5	1000
125	1.6	0.6	2.2	600
	2.0	1.0	3.0	750
	2.6	1.4	4.0	1000
	3.2	1.8	5.0	1250

Other considerations

The anchor drill holes should be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of grout tube to place grout from the bottom up in the anchor holes is recommended.

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.

5.5 Design for Earthquakes

The site class for seismic site response can be taken as **Class D**. 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.

5.6 Basement Slab / Slab-on-Grade

With the removal of all topsoil and deleterious fill from within the footprint of the proposed buildings, the native soil surface or approved engineered fill surface will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

For the single-storey additions with slab-on-grade construction, the upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

For structures with basement slabs, it is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone. Further, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided underlying the basement slabs. This is discussed further in Subsection 6.1.

If a raft slab is considered, 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.

5.7 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/m^3)
H = height of the wall (m)

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

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

Seismic Earth Pressures

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

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

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

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

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.8 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 6 and 7.

Table 6 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 7 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

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

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 understood 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 2 m below finished grade down the foundation walls to the bottom of foundation.

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 grade beam). 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, grade beam, 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.

It is also recommended that a perimeter foundation drainage system be provided at a depth of 2 m for the proposed structure to control any surficial groundwater. The perimeter drainage pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration for any basement 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, 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, unheated structures such as the access ramp may require insulation against the deleterious effect of frost action.

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 structure.

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 is anticipated to be required to support the overburden soils for the underground parking levels. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

The temporary shoring system is recommended to consist of steel sheet piles along the west boundary of the excavation for the underground parking levels, which would be located in close proximity to the existing structures on-site. The remainder of the temporary shoring system could consist of either soldier piles and lagging or steel sheet piles. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is the preferred method.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 8 - 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.

Underpinning of Adjacent Structures

If the footings of the proposed additions, particularly the single-storey addition with a basement level, extend below the adjacent footings of the existing structures, underpinning of these footings would be required. It is anticipated that conventional concrete underpinning piers would be suitable for this project. The depth of the underpinning will be dependent on the depth of the adjacent foundations relative to the foundation depths of the proposed additions at the subject site.

It is recommended that additional test pits be completed at the subject site to evaluate the depths of the adjacent building footings in the vicinity of the proposed additions.

Excavation Base Stability

The base of supported excavations can fail by three (3) general modes:

- ☐ Shear failure within the ground caused by inadequate resistance to loads imposed by grade difference inside and outside of the excavation,
- ☐ Piping from water seepage through granular soils, and

☐ Heave of layered soils due to water pressures confined by intervening low permeability soils.

The potential for base heave in cohesive soils should be determined for stability of flexible retaining systems.

The factor of safety with respect to base heave, FS_b, is:

$$FS_b = N_b s_u / \sigma_z$$

where:

N_b - stability factor dependent upon the geometry of the excavation and given in Figure 1 on the following page.
s_u - undrained shear strength of the soil below the base level
σ_z - total overburden and surcharge pressures at the bottom of the excavation

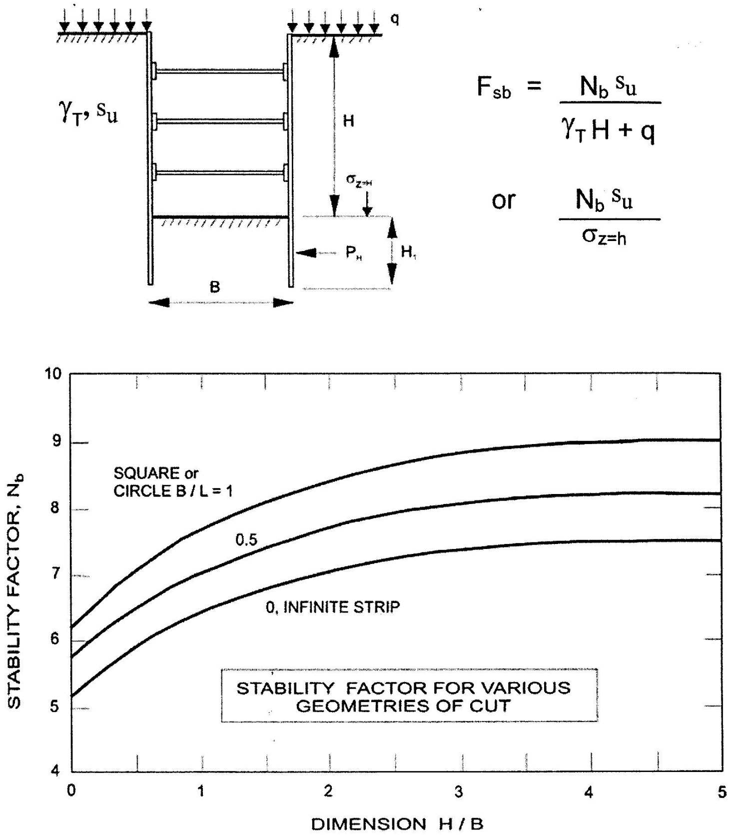

Figure 1 - Stability Factor for Various Geometries of Cut

In the case of soft to firm clays, a factor of safety of 2 is recommended for base stability.

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. Where the bedding is located within the firm grey silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 95% 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 95% of its SPMDD.

Generally, it should be possible to re-use the moist (not wet) brown 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.

To reduce long-term lowering of the groundwater at this site, clay seals should be provided within the service trenches excavated through the silty clay deposit. The seals should be at least 1.5 m long (in the trench direction) and should extend from trench wall to trench wall. The seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches excavated through the silty clay deposit.

6.5 Groundwater Control

As noted above, it is recommended that the west side of the excavation for the underground parking levels be supported using steel sheet piles to limit dewatering of the existing structures on-site. Therefore, within the excavation it is anticipated that groundwater should be controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the bottom of the excavation. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

A temporary Ministry of 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 groundwater flow will be very low to negligible. A more accurate estimate of groundwater flow can be provided at the time of construction.

Impacts on Neighbouring Properties

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 moderate to slightly aggressive corrosive environment.

6.8 Tree Planting Restrictions

The following tree planting setbacks are recommended for the subject site.

Large trees (mature height over 14 m) can be planted within this area provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits are 7.5 m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m) provided that the following conditions are met:

☐ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the centre of the tree trunk and verified by means of the Grading Plan as indicated procedural changes below.

☐ A small tree must be provided with a minimum of 25 m^3 of available soil volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.

☐ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

☐ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).

☐ Grading surround the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree), as noted on the subdivision Grading Plan.

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.
- ☐ 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 Waterford Ottawa Seniors Residence 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.

Scott Dennis

Scott S. Dennis, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Jan. 15, 2020
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

David J. Gilbert, P.Eng.

Report Distribution

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

UNIDIMENSIONAL CONSOLIDATION TESTING

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Addition - 2425 Bank Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 November 26

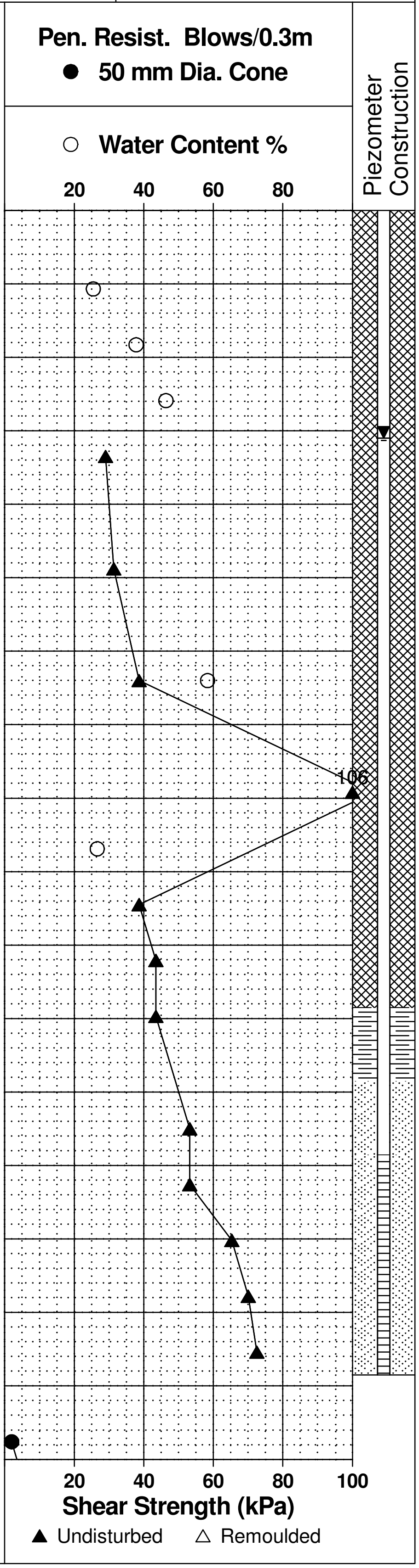
FILE NO. PG5163

HOLE NO. BH 1-19

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
FILL: Crushed stone, trace sand and gravel	0.51
TOPSOIL	0.91
Very stiff to stiff, brown **SILTY CLAY,** trace sand	2.90
Firm to stiff, grey **CLAYEY SILT** to **SILTY CLAY**	15.85
Dynamic Cone Penetration Test commenced at 15.85m depth.	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	79	9
SS	3	83	7
SS	4	71	2
SS	5	100	P
SS	6	100	W
SS	7	100	W
SS	8	100	W

DEPTH (m)	ELEV. (m)
0	89.37
1	88.37
2	87.37
3	86.37
4	85.37
5	84.37
6	83.37
7	82.37
8	81.37
9	80.37
10	79.37
11	78.37
12	77.37
13	76.37
14	75.37
15	74.37
16	73.37
17	72.37



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Addition - 2425 Bank Street
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5163

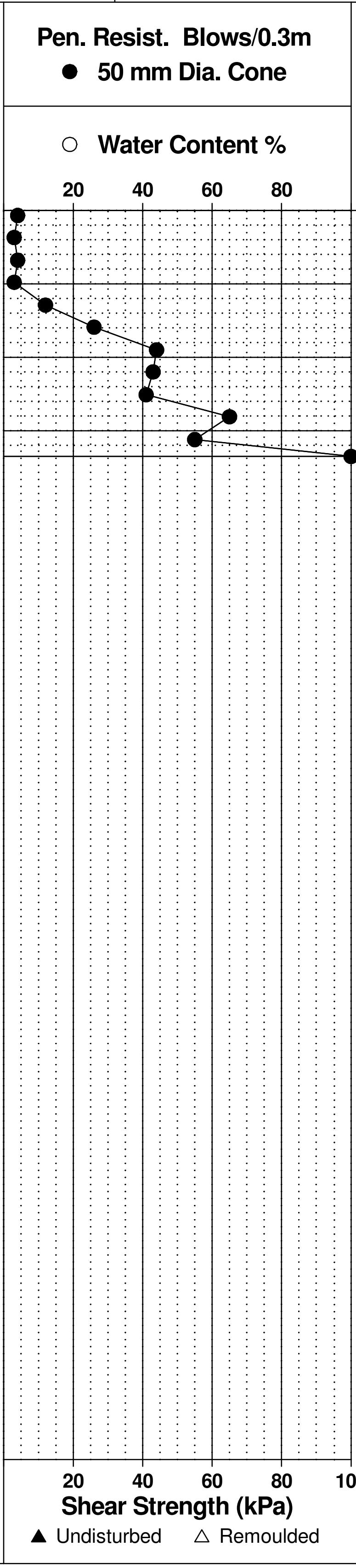
REMARKS

HOLE NO. BH 1-19

BORINGS BY CME 55 Power Auger

DATE 2019 November 26

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE								
						17	72.37	
						18	71.37	
						19	70.37	
						20	69.37	
20.35								
End of Borehole Practical DCPT refusal at 20.35m depth. (GWL @ 3.10m - Dec. 3, 2019)								



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Addition - 2425 Bank Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

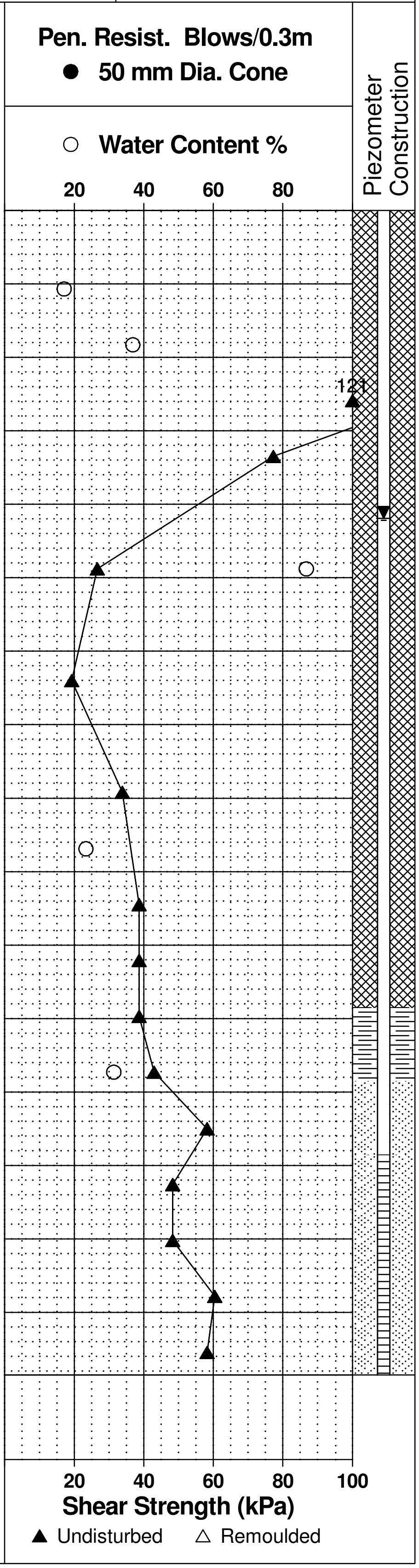
DATE 2019 November 26

FILE NO. PG5163

HOLE NO. BH 2-19

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.04	AU	1		
FILL: Brown sand to silty sand with gravel, crushed stone	1.22	SS	2	62	10
Very stiff to stiff, brown **SILTY CLAY**	3.50	SS	3	92	9
Firm, grey **CLAYEY SILT**	15.85	SS	4	100	W
		SS	5	54	W
		SS	6	100	P
		SS	7	100	W
End of Borehole					
(GWL @ 4.19m - Dec. 3, 2019)					

DEPTH (m)	ELEV. (m)
0	89.78
1	88.78
2	87.78
3	86.78
4	85.78
5	84.78
6	83.78
7	82.78
8	81.78
9	80.78
10	79.78
11	78.78
12	77.78
13	76.78
14	75.78
15	74.78



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Addition - 2425 Bank Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 November 27

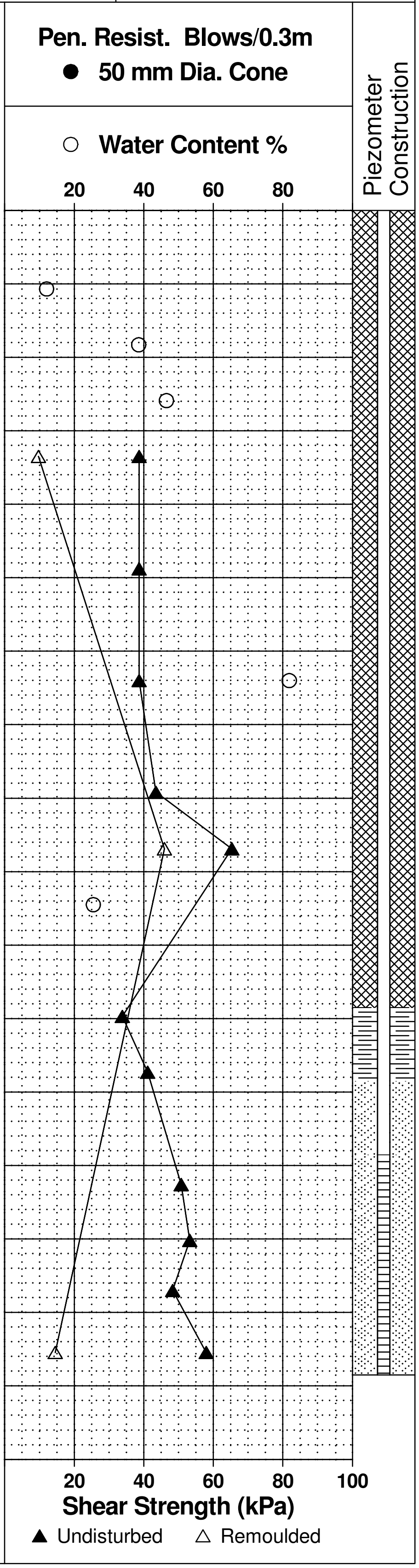
FILE NO. PG5163

HOLE NO. BH 3-19

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.05
FILL: Crushed stone, trace sand	0.33
FILL: Brown silty sand with gravel and crushed stone	1.22
Very stiff to stiff, brown **SILTY CLAY,** some sand	3.00
Firm to stiff, grey **CLAYEY SILT**	15.85
Dynamic Cone Penetration Test commenced at 15.85m depth. Cone pushed to 17.4m depth.	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	17	13
SS	3	88	7
SS	4	100	3
SS	5	100	P
SS	6	100	W
SS	7	100	W
SS	8	100	W

DEPTH (m)	ELEV. (m)
0	90.12
1	89.12
2	88.12
3	87.12
4	86.12
5	85.12
6	84.12
7	83.12
8	82.12
9	81.12
10	80.12
11	79.12
12	78.12
13	77.12
14	76.12
15	75.12
16	74.12
17	73.12



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Addition - 2425 Bank Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

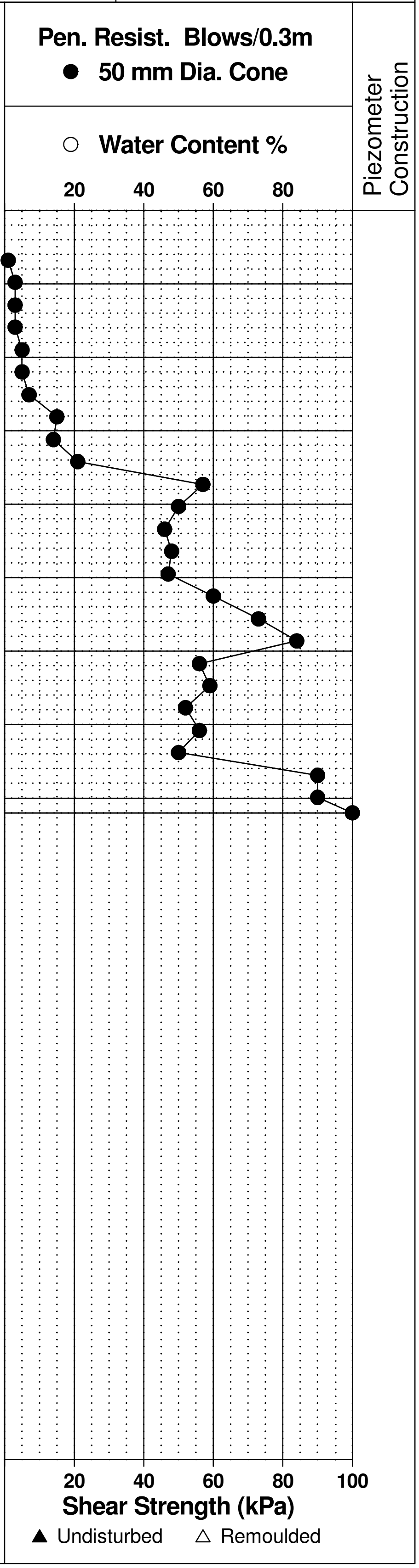
BORINGS BY CME 55 Power Auger

DATE 2019 November 27

FILE NO. PG5163

HOLE NO. BH 3-19

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						17	73.12
	▨					18	72.12
	▨					19	71.12
	▨					20	70.12
	▨					21	69.12
	▨					22	68.12
	▨					23	67.12
	▨					24	66.12
25.20	▨					25	65.12
End of Borehole Practical DCPT refusal at 25.20m depth.							



Piezometer Construction

JOHN D. PATERSON & ASSOCIATES LTD.
Consulting Geotechnical and Environmental Engineers
28 Concourse Gate, Nepean, Ont. K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Southway Inn Addition, 2431 Bank St.
Ottawa, Ontario

DATUM TBM - Top of floor slab @ main entrance to existing hotel reception. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 23 September 1999

FILE NO. G7535

HOLE NO. BH 1

SOIL DESCRIPTION

GROUND SURFACE

Soil description	Depth (m)
Asphaltic concrete	0.13
FILL: Brown silty sand, occasional gravel	1.07
	2.13
Loose, brown **SILTY SAND/SANDY SILT** interlayered with clayey silt	
Firm to soft, grey **CLAYEY SILT**, trace of sand	18.50
Inferred **TILL** deposits, increasing cobbles and boulders with depth.	20.11

End of Borehole

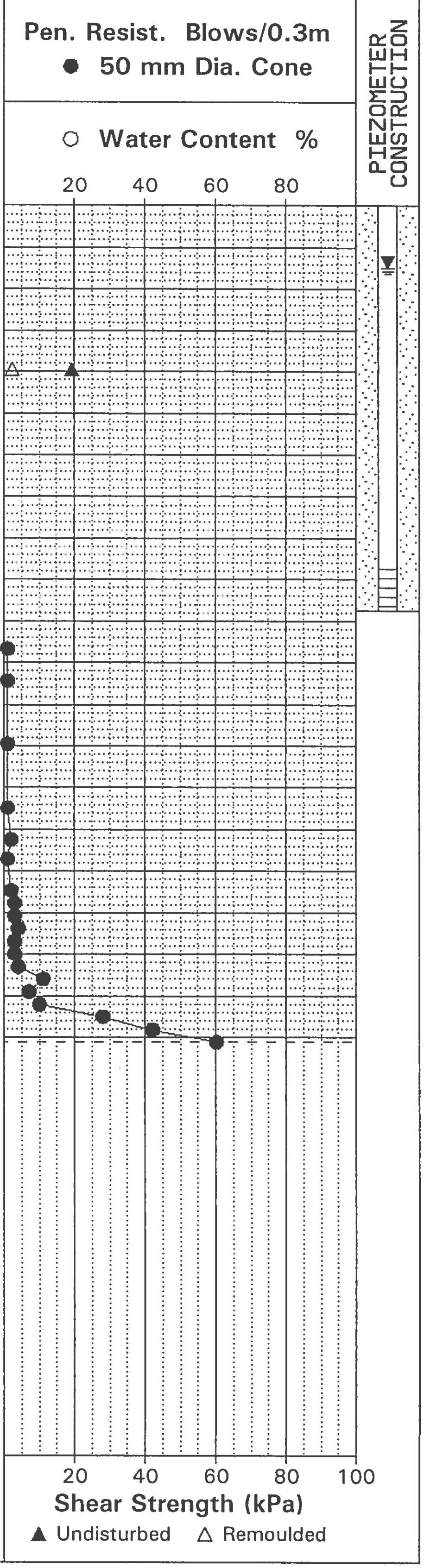
Dynamic Cone Penetration Test commenced @ 9.75m depth.

Cone refusal on possible bedrock @ 20.11m depth.

(GWL @ 1.5m-Oct. 1/99)

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	1	42	4
SS	2	100	4
SS	3	100	1
SS	4	100	1
SS	5	100	1
SS	6	100	1
SS	7	100	1

DEPTH (m)	ELEV. (m)
0	99.48
1	98.48
2	97.48
3	96.48
4	95.48
5	94.48
6	93.48
7	92.48
8	91.48
9	90.48
10	89.48
11	88.48
12	87.48
13	86.48
14	85.48
15	84.48
16	83.48
17	82.48
18	81.48
19	80.48
20	79.48



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SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Southway Inn Addition, 2431 Bank St.
Ottawa, Ontario

DATUM TBM - Top of floor slab @ main entrance to existing hotel reception. Assumed elevation = 100.00m.

REMARKS

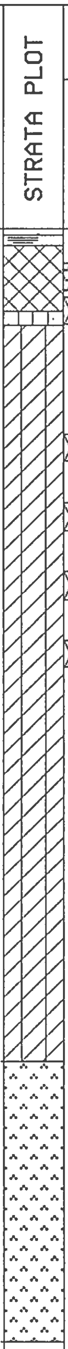
BORINGS BY CME 55 Power Auger

DATE 23 September 1999

FILE NO. G7535

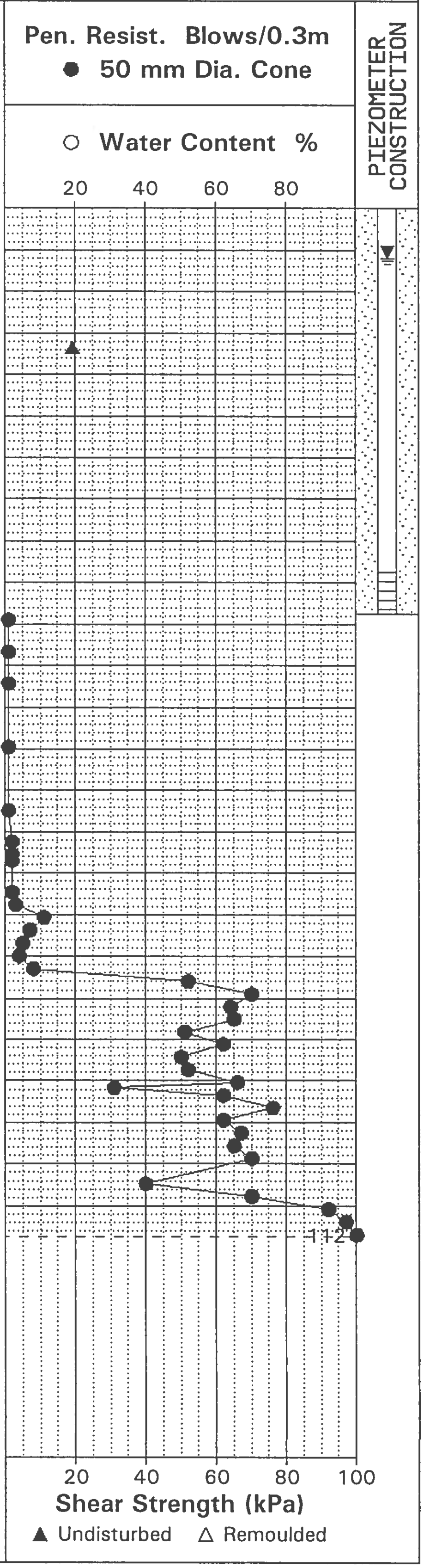
HOLE NO. BH 2

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.14
Crushed stone	0.36
FILL: Black silty sand	1.83
Loose, grey to brown **SILTY SAND**	2.13
Firm to soft, grey CLAYEY SILT, trace of sand	18.50
Inferred TILL deposits with cobbles and probable boulders	24.74
End of Borehole	



SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	8		
SS	9	100	2
SS	10	100	1
SS	11	100	1
SS	12	100	8
SS	13	100	1

DEPTH (m)	ELEV. (m)
0	99.18
1	98.18
2	97.18
3	96.18
4	95.18
5	94.18
6	93.18
7	92.18
8	91.18
9	90.18
10	89.18
11	88.18
12	87.18
13	86.18
14	85.18
15	84.18
16	83.18
17	82.18
18	81.18
19	80.18
20	79.18
21	78.18
22	77.18
23	76.18
24	75.18



Dynamic Cone Penetration Test commenced @ 9.75m depth.
Cone refusal on inferred bedrock @ 24.74m depth.

(GWL @ 1.2m-Oct. 1/99)

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SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Southway Inn Addition, 2431 Bank St.
Ottawa, Ontario

DATUM TBM - Top of floor slab @ main entrance to existing hotel reception. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 23 September 1999

FILE NO. **G7535**

HOLE NO. **BH 3**

SOIL DESCRIPTION	
GROUND SURFACE	
Asphaltic concrete	0.11
Crushed stone	0.43
FILL: Brown silty sand	0.76
	0.91
Dark brown sandy **TOPSOIL**	1.52
	2.13
Loose, brown **SANDY SILT**	
Interlayered brown **SILTY SAND** and **SILTY CLAY**	
Firm to soft, grey **CLAYEY SILT**	18.30
Inferred **TILL** deposits, increasing cobbles and probable boulders with depth.	21.79

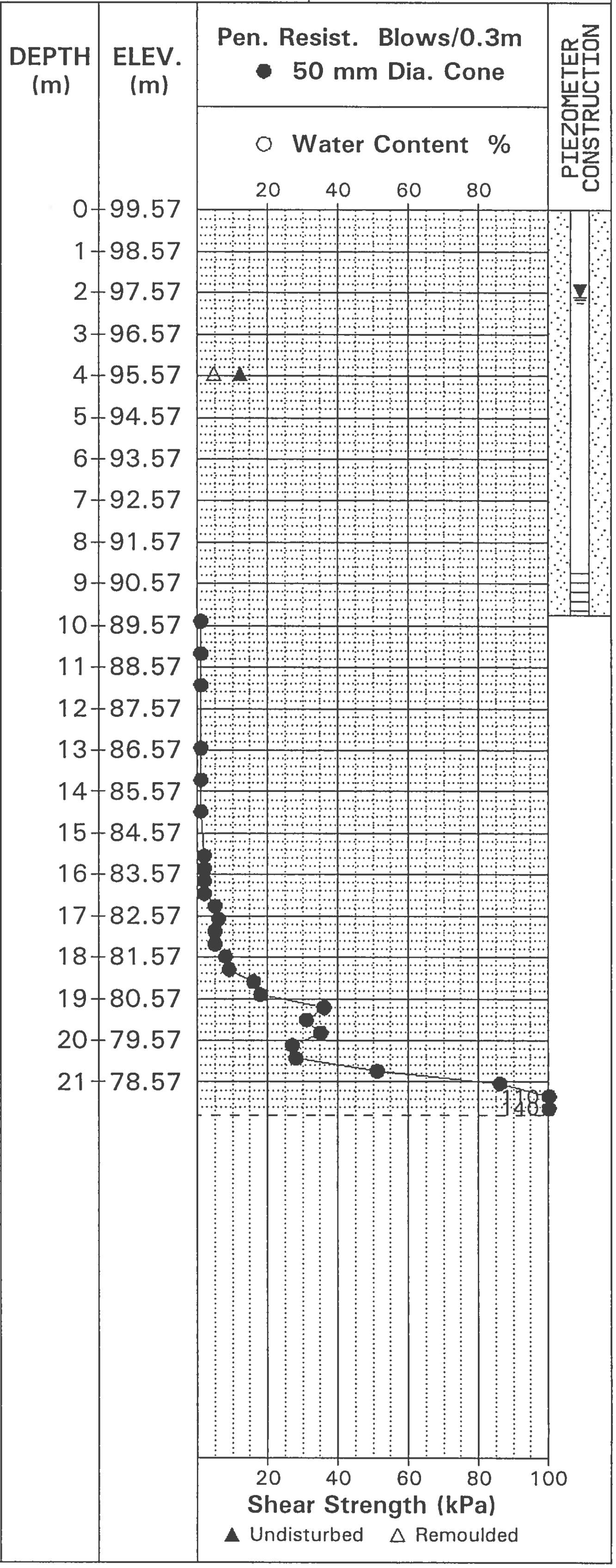
End of Borehole

Dynamic Cone Penetration Test commenced @ 9.75m depth.

Cone refusal on possible bedrock @ 21.79m depth

(GWL @ 2.1m-Oct. 1/99)

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
SS	14	83	6
SS	15	83	3
SS	16	100	1
SS	17	100	1
SS	18	100	1
SS	19	100	1
SS	20	100	1



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SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Southway Inn Addition, 2431 Bank St.
Ottawa, Ontario

DATUM TBM - Top of floor slab @ main entrance to existing hotel reception. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 24 September 1999

FILE NO. **G7535**

HOLE NO. **BH 4**

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
Topsoil	0.10				
FILL: Dark brown to brown silty sand, occasional brick pieces		AU	21		
		SS	22	83	3
	3.05				
Firm to soft, grey **CLAYEY SILT**, trace of sand		SS	23	100	1
		SS	24	100	1
		SS	25	100	1
		SS	26	100	1
	16.76				

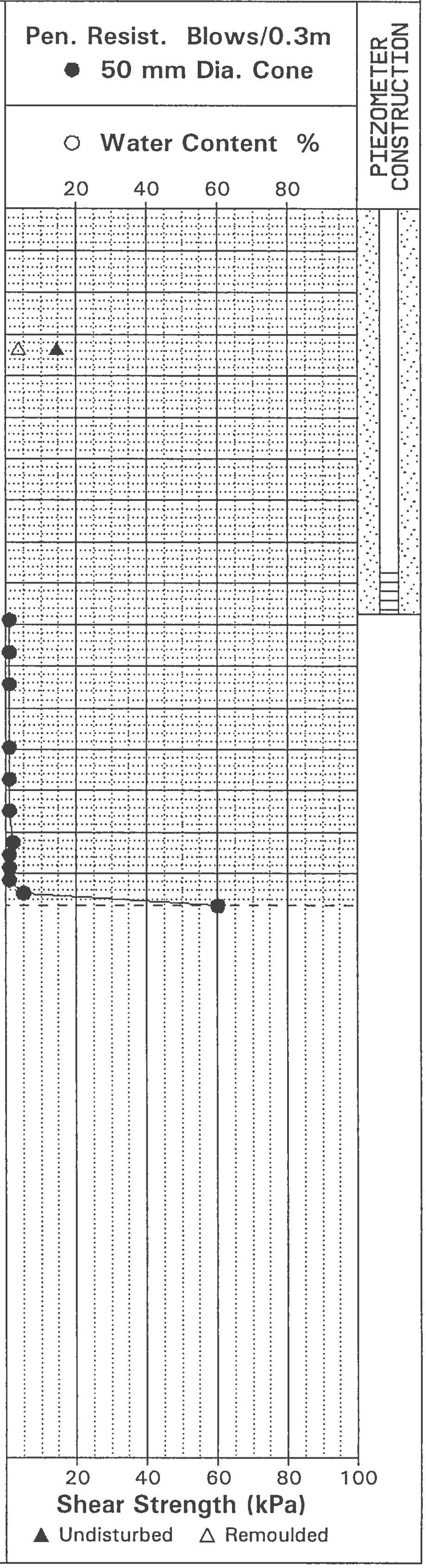
End of Borehole

Dynamic Cone Penetration Test commenced @ 9.75m depth.

Cone refusal on possible bedrock @ 16.76m depth.

(Piezometer blocked-Oct. 1/99)

DEPTH (m)	ELEV. (m)
0	99.95
1	98.95
2	97.95
3	96.95
4	95.95
5	94.95
6	93.95
7	92.95
8	91.95
9	90.95
10	89.95
11	88.95
12	87.95
13	86.95
14	85.95
15	84.95
16	83.95



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SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Southway Inn Addition, 2431 Bank St.
Ottawa, Ontario

DATUM TBM - Top of floor slab @ main entrance to existing hotel reception. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 24 September 1999

FILE NO. **G7535**

HOLE NO. **BH 5**

SOIL DESCRIPTION		STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	PIEZOMETER CONSTRUCTION
			TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE							0	99.09		
Topsoil	0.13		AU	27						
FILL: Brown silty sand	0.76		SS	28	92	6	1	98.09		
Loose, brownish grey **SANDY SILT**	1.37									
End of Borehole (BH dry upon completion)										

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Southway Inn Addition, 2431 Bank St.
Ottawa, Ontario

DATUM TBM - Top of floor slab @ main entrance to existing hotel reception. Assumed elevation = 100.00m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 24 September 1999

FILE NO. **G7535**

HOLE NO. **BH 6**

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
Topsoil	0.15	AU	29			0	99.34
FILL: Brown silty sand	0.60	SS	30	83	9	1	98.34
Dark brown sandy **TOPSOIL**	0.86						
Loose, brown **SANDY SILT**	1.37						
End of Borehole							
(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

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SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Southway Inn Addition, 2431 Bank St.
Ottawa, Ontario

DATUM TBM - Top of floor slab @ main entrance to existing hotel reception. Assumed elevation = 100.00m.

REMARKS

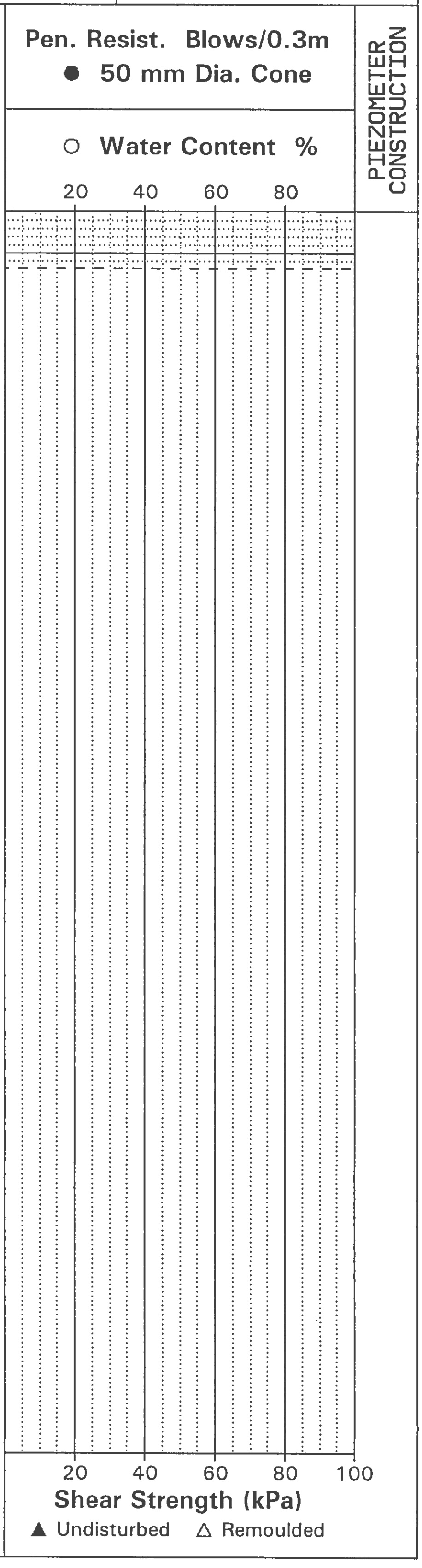
BORINGS BY CME 55 Power Auger

DATE 24 September 1999

FILE NO. **G7535**

HOLE NO. **BH 7**

SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							0	99.89
Topsoil	0.13		AU	31				
FILL: Brown silt and sand, some organics	1.22		SS	32	92	6	1	98.89
Dark brown sandy silt **TOPSOIL**	1.37							
End of Borehole								
(BH dry upon completion)								



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SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Southway Inn Addition, 2431 Bank St.
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY CME 55 Power Auger

DATE 23 March 1900

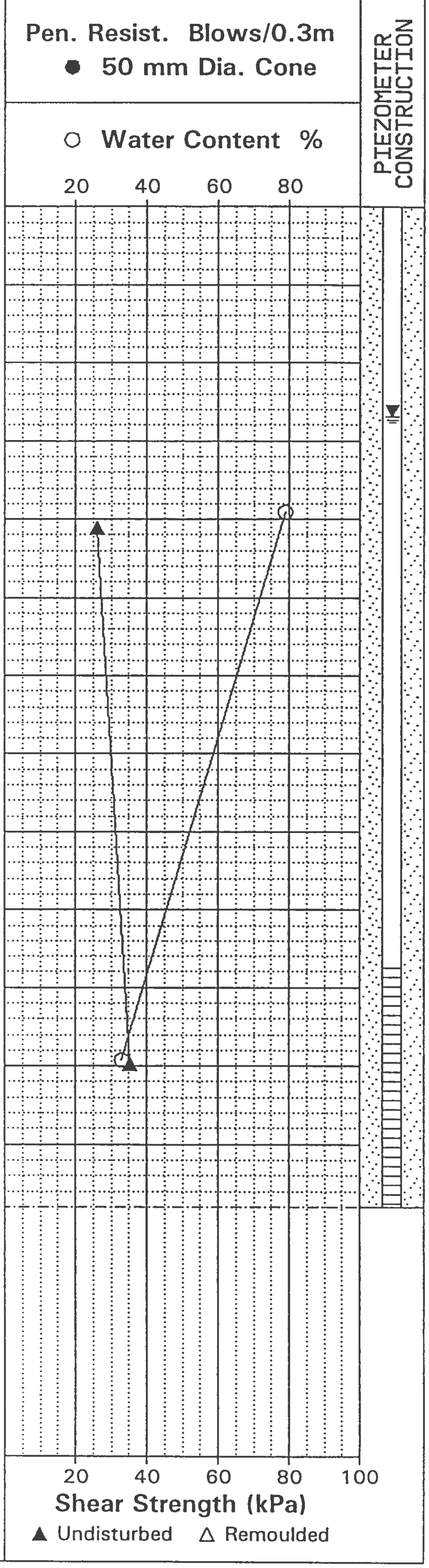
FILE NO. G7535

HOLE NO. BH 8

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Topsoil	0.15
FILL: Dark brown silty sand, occasional gravel	1.22
Loose, brown **SILTY SAND**	1.98
Firm to soft, grey **CLAYEY SILT**, trace sand, sand seams and layers	12.80
End of Borehole	
(GWL @ 2.69m-March 27/00)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	100	5
SS	3	50	3
TW	1		
TW	2		
TW	3		
TW	4		
TW	5		
TW	6		

DEPTH (m)	ELEV. (m)
0	99.39
1	98.39
2	97.39
3	96.39
4	95.39
5	94.39
6	93.39
7	92.39
8	91.39
9	90.39
10	89.39
11	88.39
12	87.39



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SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Southway Inn Addition, 2431 Bank St.
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY Backhoe

DATE 22 November 1999

FILE NO. **G7535**

HOLE NO. **TP 1**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % (20, 40, 60, 80)	PIEZOMETER CONSTRUCTION
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE						0			
Dark brown silty sand **TOPSOIL** 0.20									
Loose, reddish brown **SAND**, some silt 1.40		G	1			1			
Stiff, grey, dessicated **SILTY CLAY** 2.29						2			
End of Test Pit									

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

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SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Southway Inn Addition, 2431 Bank St.
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY Backhoe

DATE 22 November 1999

FILE NO. **G7535**

HOLE NO. **TP 2**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	PIEZOMETER CONSTRUCTION
GROUND SURFACE						0			
Topsoil (fill) 0.13									
FILL: Grey to brown silty sand-gravel 1.22		G	2			1			
Loose, reddish brown **SAND**, some silt 1.83									
Stiff, grey, dessicated **SILTY CLAY** 2.44						2			
End of Test Pit									

Water Content %: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Southway Inn Addition, 2431 Bank St.
Ottawa, Ontario

DATUM

REMARKS

BORINGS BY Backhoe

DATE 22 November 1999

FILE NO. **G7535**

HOLE NO. **TP 3**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	
Topsoil (fill) 0.08							
FILL: Brown silty clay with some sand, gravel, boulders and asphalt fragments		G	3			1	
		G	4				
1.98						2	
Black, organic silty clay **TOPSOIL** 2.13							
Stiff, grey SILTY CLAY 2.74							
End of Test Pit							

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

Cc - Concavity coefficient = $(D30)^2$ / (D10 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)

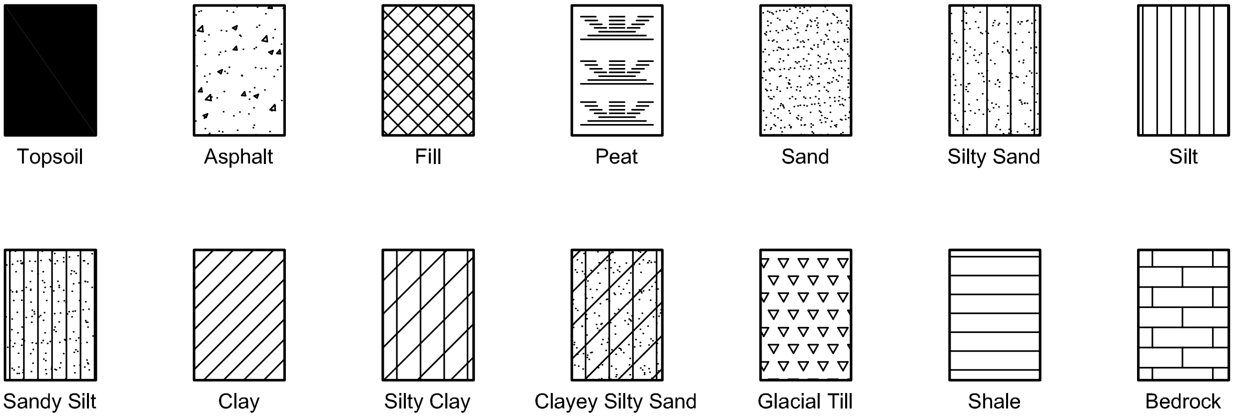
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

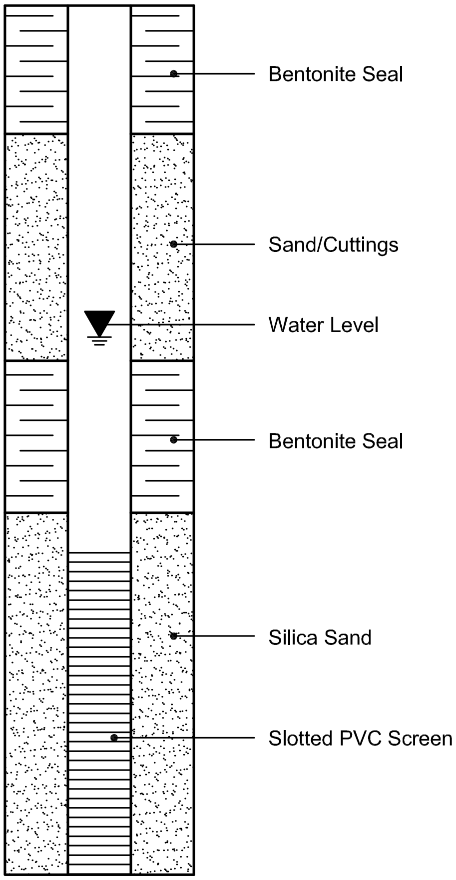
SYMBOLS AND TERMS (continued)

STRATA PLOT

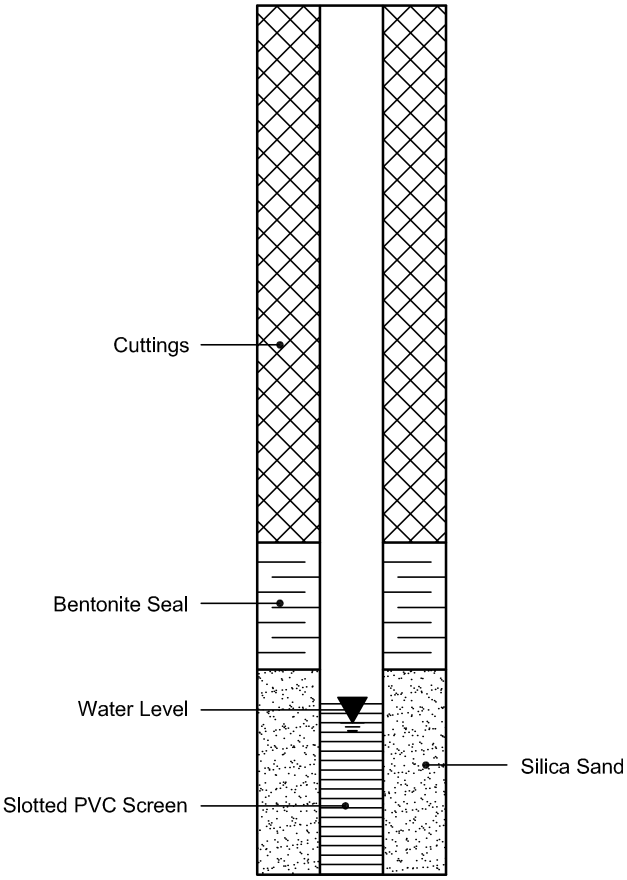


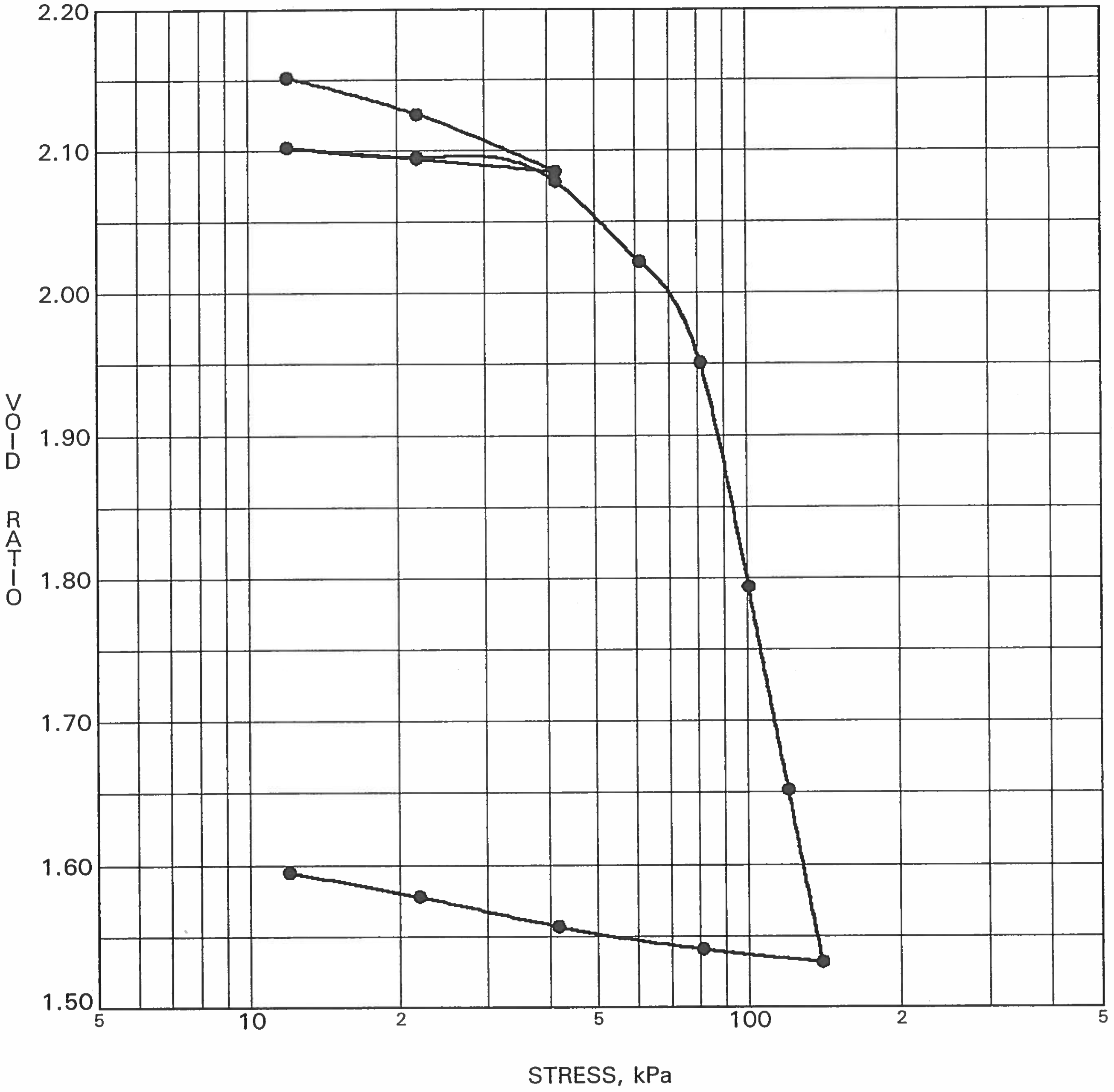
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 8	p'_o	30 kPa	Ccr	0.038
Sample No.	TW 1	p'_c	75 kPa	Cc	2.846
Sample Depth	3.91 m	OC Ratio	2.5	Wo	78.7 %
Sample Elev.	95.48 m	Void Ratio	2.176	Unit Wt.	15.0 kN/m^3

CLIENT	Mr. Bill Zlepnig	FILE NO.	G7535
PROJECT	Geotechnical Investigation - Proposed Southway Inn Addition, 2431 Bank St.	DATE	27/03/00

CONSOLIDATION TEST

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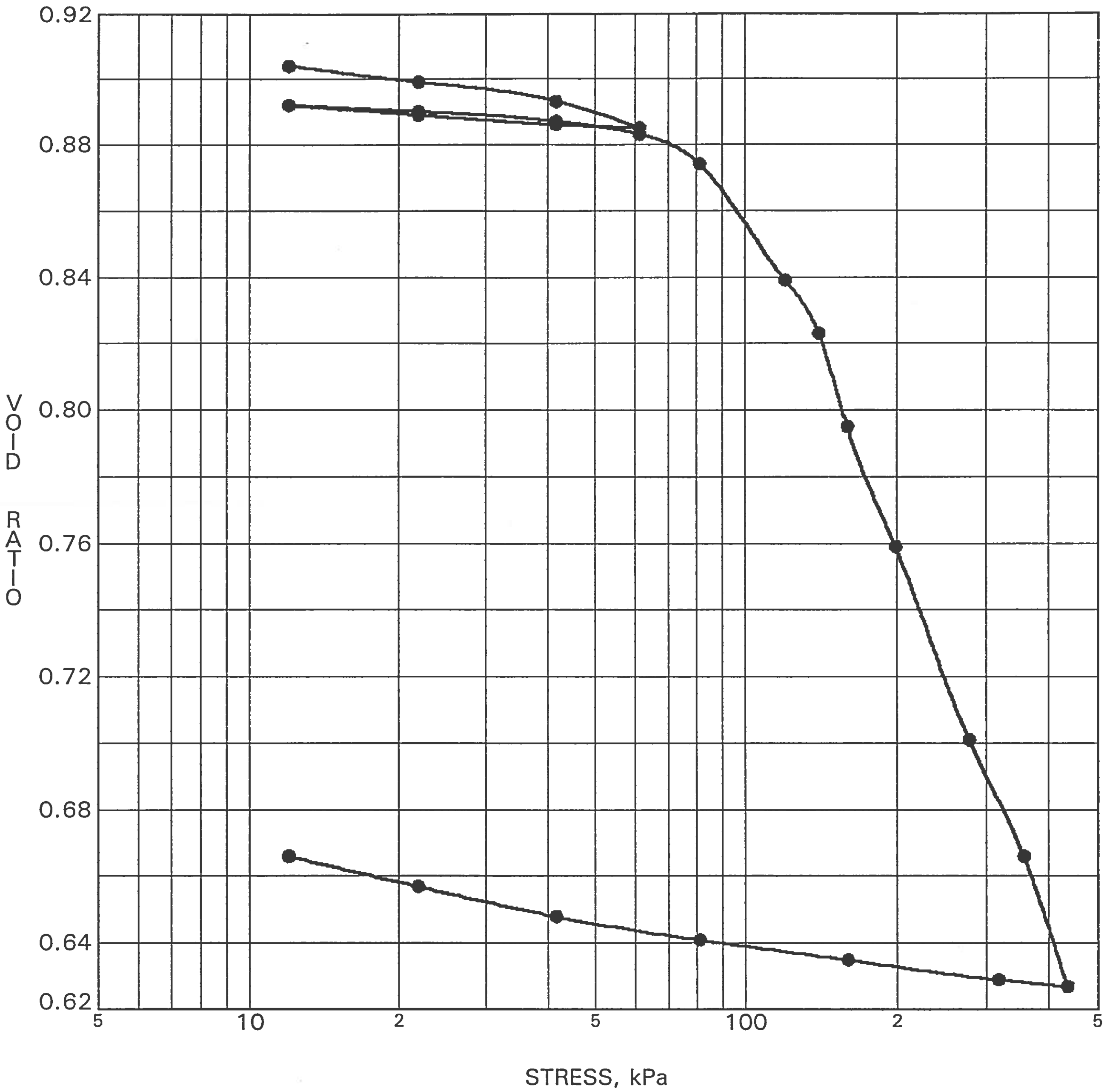
Unit 1, 28 Concourse Gate, Nepean, Ontario K2E 7T7



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 8	p'_o	68 kPa	Ccr	0.012
Sample No.	TW 5	p'_c	100 kPa	Cc	0.375
Sample Depth	10.92 m	OC Ratio	1.5	Wo	32.9 %
Sample Elev.	88.47 m	Void Ratio	0.910	Unit Wt.	15.0 kN/m^3

CLIENT **Mr. Bill Zlepnig**

PROJECT **Geotechnical Investigation - Proposed**
Southway Inn Addition, 2431 Bank St.

FILE NO. **G7535**

DATE **27/03/00**

CONSOLIDATION TEST

JOHN D. PATERSON & ASSOCIATES LTD.
Unit 1, 28 Concourse Gate, Nepean, Ontario K2E 7T7

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29200

Report Date: 03-Dec-2019

Order Date: 27-Nov-2019

Project Description: PG5163

	Client ID:	BH2-19, SS3	-	-	-
	Sample Date:	26-Nov-19 14:00	-	-	-
	Sample ID:	1948382-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	76.6	-	-	-
General Inorganics					
pH	0.05 pH Units	7.56	-	-	-
Resistivity	0.10 Ohm.m	5.63	-	-	-
Anions					
Chloride	5 ug/g dry	745	-	-	-
Sulphate	5 ug/g dry	783	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5163-1 - TEST HOLE LOCATION PLAN

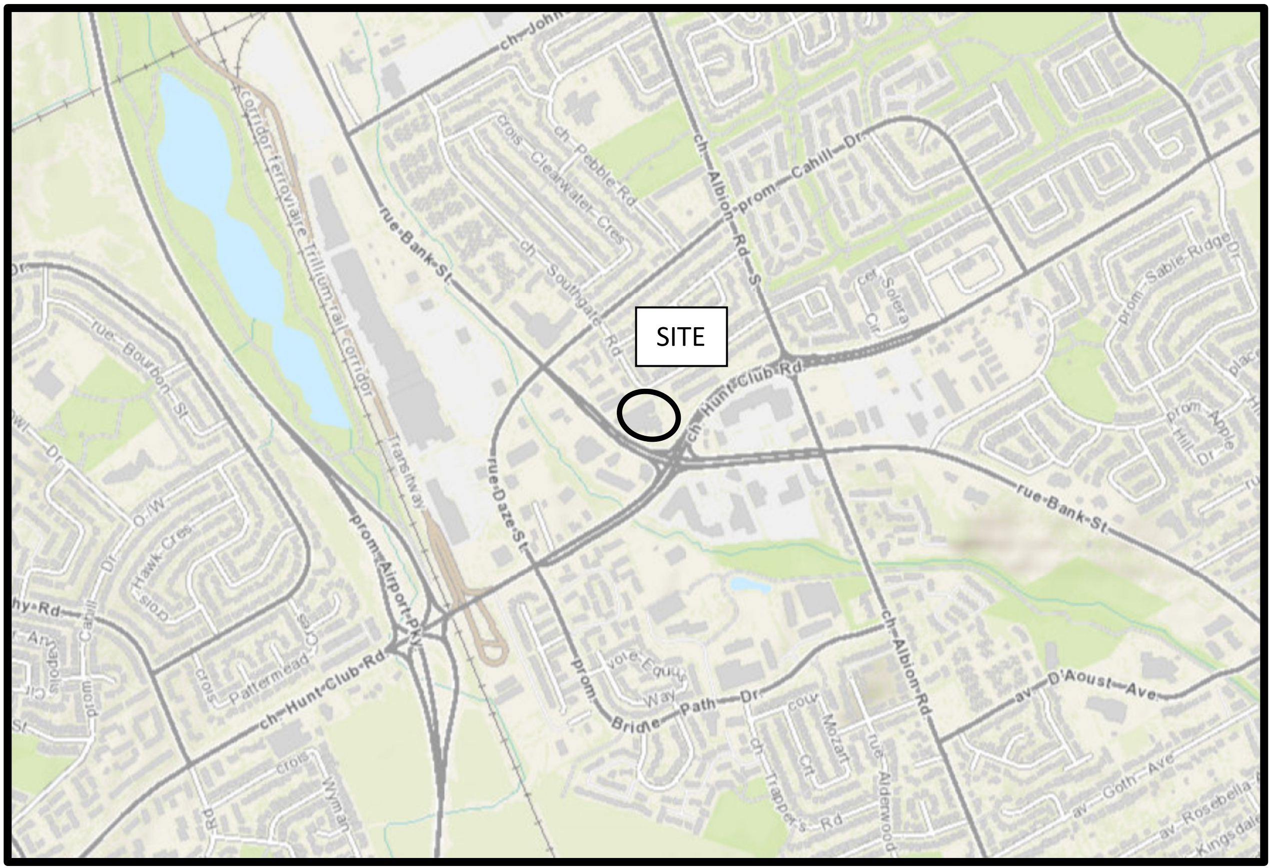
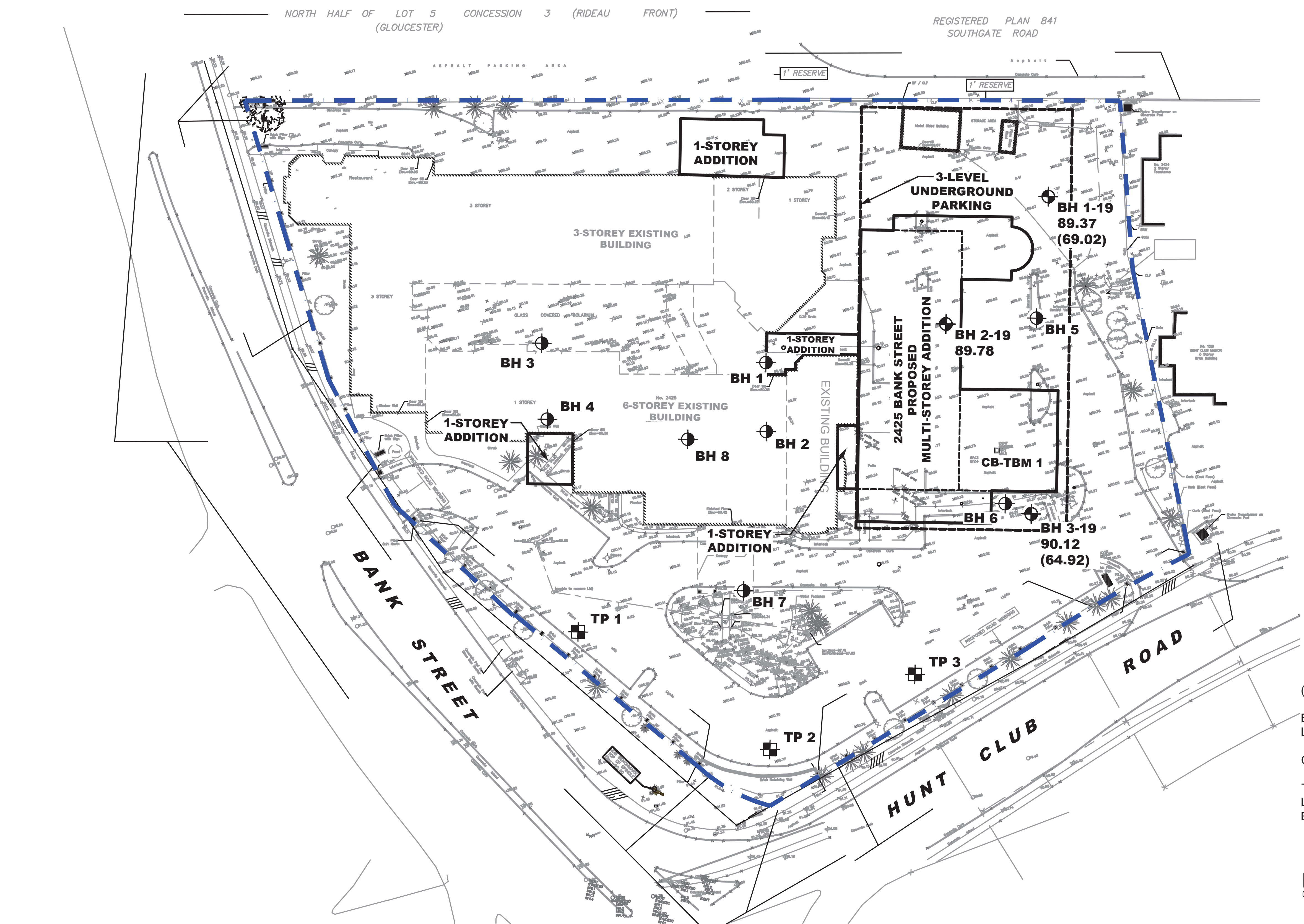


FIGURE 1

KEY PLAN



LEGEND:

⊕	BOREHOLE LOCATION (CURRENT INVESTIGATION)
●	BOREHOLE LOCATION (PATERSON GROUP REPORT, G7535, SEPTEMBER 1999)
▣	TEST PIT LOCATION (PATERSON GROUP REPORT, G7535, SEPTEMBER 1999)
89.37	GROUND SURFACE ELEVATION (m)
(68.75)	PRACTICAL REFUSAL TO DCPT ELEVATION (m)

BASE PLAN PREPARED BY ANNIS, O'SULLIVAN, VOLLEBEKK LIMITED.

CONCEPTUAL PLAN PROVIDED BY NEUF ARCHITECTS.

TBM 1- TOP OF GRATE OF EXISTING CATCH BASIN (CBMH4) LOCATED IN PARKING LOT ON SUBJECT SITE. GEODETIC ELEVATION = 89.65 m

SCALE: 1:750

0 10 20 30 50m

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL

WATERFORD OTTAWA SENIORS RESIDENCE
GEOTECHNICAL INVESTIGATION
2425 BANK STREET
PROPOSED MULTI-STOREY ADDITION

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750
Date: 12/2019
Drawn by: YA
Report No.: PG5163-1
Checked by: RG
Dwg. No.: PG5163-1
Approved by: DJG
Revision No.:

p:\autocad drawings\geotechnical\pg51xx\pg5163\pg5163-1-test hole location plan(rev).dwg