patersongroup

Consulting Engineers

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

Geotechnical Engineering
Environmental Engineering
Hydrogeology
Geological Engineering
Materials Testing
Building Science
Archaeological Studies

www.patersongroup.ca

November 20, 2012
File: PG2779-LET.01R

Robertson Martin Architects
216 Pretoria Avenue
Ottawa, Ontario
L1S 1X2

Attention: **Mr. Robert Martin**

Subject: **Geotechnical Investigation**
Proposed Perimeter Wall Rehabilitation
West Slope - Parliament Hill - Ottawa

Dear Sir,

Paterson Group (Paterson) has prepared the following letter report to present our findings from our geotechnical investigation and address outstanding issues from a geotechnical perspective for the proposed perimeter wall rehabilitation to be completed along the west slope of Parliament Hill, in the City of Ottawa, Ontario. The following letter report presents our findings and recommendations.

1.0 Field Investigation

The fieldwork for our geotechnical investigation was conducted on October 26, 2012, and consisted of extending three (3) boreholes using portable drilling equipment. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer from the geotechnical division.

The location and ground surface elevations at the borehole locations were surveyed by Paterson field personnel. Ground surface elevations at the borehole locations were referenced to a temporary benchmark (TBM), consisting of the manhole adjacent to the south stairs. A geodetic elevation of 85.50 m was provided for the TBM. The location of the TBM, test holes and the ground surface elevations of the test hole locations are presented on Drawing PG2779-1 - Test Hole Location Plan attached to the present letter.

2.0 Field Observations

The subsurface profile encountered at the borehole locations consists of a 50 mm thick layer of asphaltic concrete or silty sand fill overlying fill, consisting of silty sand with mortar, brick, crushed stone, gravel and cobbles. Practical split spoon refusal was encountered at all borehole locations at depths varying between 2.4 to 3 m. Reference should be made to the Soil Profile and Test Data sheets attached to the present letter for specific details of the soil profile encountered at the test hole locations.

All boreholes were observed to be dry upon completion of the sampling program. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

Also, based on available geological mapping, bedrock consists of limestone of the Lindsay Formation and is expected to range between 0 and 5 m depth in the area of the subject site.

Based on observations within the exploratory openings completed along the base of the existing wall, it was noted that the existing wall is founded over an approximately 150 to 250 mm thick concrete pad. The concrete pad was noted to consist of a mixture of cement and poorly graded cobbles. The majority of the pad was intact, but deteriorated at the time of excavation. The subgrade material below the concrete pad consisted of a silty sand with gravel and cobble fill material. However, it is understood that bedrock was encountered approximately 0.6 to 0.7 m below existing ground surface within the exploratory opening at Pier 48.

3.0 Geotechnical Assessment

Existing Wall Assessment

Based on our observations and age of the perimeter wall, the founding conditions for the majority of the perimeter wall were functioning adequately. The majority of the wall was noted to be vertical and a tight seam between the asphaltic concrete pathway and wall was noted. However, the south portion of the existing wall in the area of the cat colony is overturning to the east. It is suspected that the overturning action is partly due to insufficient footing depth, footing size and significant frost action from the increased foot traffic in the immediate area of the cat colony.

Proposed Perimeter Wall Recommendations

It is understood that a shallow footing is anticipated as the foundation for the perimeter wall structure. It is further understood that the footing size will be increased and a rock anchor will be installed at the pier locations, which include a fall protection anchor. Based on our findings, the proposed foundation system is suitable for the subject site. It is recommended that the underside of footing be provided with at least 600 mm of soil cover and be founded over a minimum 300 mm thick layer of Granular A crushed stone. Areas with poor performing soils encountered at subgrade level should be removed and replaced with Granular A or Granular B Type II materials as noted below.

For areas where bedrock is encountered at or above subgrade level, the recommended granular fill bedding is not required. Also, it is anticipated that additional frost protection is not required for footings placed over a bedrock bearing surface provided the bedrock is free of significant fractures and mud seams.

It is anticipated that the proposed wall structure will be designed to tolerate minimal movements associated with frost heave action (ie.- strategically located construction joints throughout the structure). Based on the soils encountered, frost heave action is not anticipated to be significant in nature and poses a limited concern for the proposed wall structure. Provided the proposed wall structure can tolerate minor movements, the abovenoted soil cover is anticipated to provide sufficient frost protection for the proposed wall structure.

Site Grading and Preparation

Asphalt, topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any settlement sensitive structures.

Fill used for grading beneath the settlement sensitive structures, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. 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 perimeter wall should be compacted to at least 98% of the 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. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of the respective SPMDD.

Foundation Design

Footings placed on existing silty sand fill, free of organics and deleterious materials, under dry conditions can be designed using a bearing resistance value at serviceability limit states (SLS) of **100 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **150 kPa**, incorporating a geotechnical resistance factor of 0.5. The settlement associated with the bearing resistance value at SLS is expected to be negligible.

An acceptable soil bearing surface consists of a surface 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 for the abovenoted soils 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 soil 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 than the native soil.

Design for Earthquakes

Foundations for the proposed wall can be designed using a seismic site response **Class C** as defined in the Ontario Building Code 2006 (OBC 2006; Table 4.1.8.4.A). The soils underlying the site are not susceptible to liquefaction.

Rock Anchor Design

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

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

It should be further noted that centre to centre spacing between bond lengths be at least four (4) times the anchor hole diameter and greater than 1.2 m to lower the group influence effects. It is also recommended that anchors in close proximity to each other be grouted at the same time to ensure any fractures or voids are completely in-filled and that fluid grout does not flow from one hole to an adjacent empty one.

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

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

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

Grout to Rock Bond

Generally, the unconfined compressive strength of limestone ranges between 60 and 120 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 0.3, can be used. 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 subsoils information, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.575 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Rock anchor lengths can be designed based on the required loads. Rock anchor lengths for some typical loads have been calculated and are presented on the following page. Load specified rock anchor lengths can be provided, if required.

For our calculations the following parameters were used.

Table 1 - 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 Limestone Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	60 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

From a geotechnical perspective, the fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 and 125 mm diameter hole are provided in Table 2.

Table 2 - 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	1.2	0.6	1.8	250
	1.9	0.8	2.7	500
	3	1.5	4.5	1000
125	1.1	0.5	1.6	250
	1.5	0.7	2.2	500
	2.6	1	3.6	1000

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further 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. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

Pavement Structure

It is anticipated that the existing asphaltic concrete finished pathway is to be reinstated upon completion of the wall rehabilitation work. The proposed pavement structure shown in Table 3 is recommended for the pathway.

Table 3 - Recommended Pavement Structure - Pedestrian Pathway	
Thickness (mm)	Material Description
50	**WEAR COURSE -** HL-3 or Superpave 12.5 Asphaltic Concrete
200	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Either in situ soil, fill or OPSS Granular B Type 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 98% of the material's SPMDD using suitable compaction equipment.

4.0 Design and Construction Precautions

Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1.5H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

A trench box is recommended to be used at all times to protect personnel working in with steep or vertical sides. Services are expected to be installed by "cut and cover" methods and excavations will not be left open for extended periods of time.

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

5.0 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:

- ❑ 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.

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

6.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. Our recommendations should be reviewed when the project drawings and specifications are complete.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request that we be notified immediately in order to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein, or by person(s) other than Robertson Martin Architects or their agents is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Richard Groniger, C. Tech.

David J. Gilbert, P.Eng.

Attachments

- ❏ Soil Profile and Test Data sheets
- ❏ Figure 1 - Key Plan
- ❏ Drawing PG2779-1 - Test Hole Location Plan

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Perimeter Wall Rehabilitation - West Slope
Parliament Hill, Ottawa, Ontario

DATUM Borehole ground surface elevations referenced to a geodetic datum.

FILE NO. PG2779

REMARKS

HOLE NO. BH 1

BORINGS BY Portable Drill **DATE** October 26, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	86.17
Asphaltic concrete 0.05							
FILL: Grey crushed stone		AU	1				
		SS	2	25	13	1	85.17
1.40							
FILL: Brown silty fine sand with mortar, brick, trace clay, gravel and glass		SS	3	50	7		
						2	84.17
		SS	4	33	8		
		SS	5	69	50+		
2.95							
End of Borehole Practical spoon refusal at 2.95m depth							

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

○ Water Content %

20 40 60 80

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

Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Perimeter Wall Rehabilitation - West Slope
Parliament Hill, Ottawa, Ontario

DATUM Borehole ground surface elevations referenced to a geodetic datum.

FILE NO. PG2779

REMARKS

HOLE NO. BH 2

BORINGS BY Portable Drill **DATE** October 26, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	85.99		
Asphaltic concrete (0.05)									
FILL: Grey crushed stone (0.69)		AU	1						
FILL: Grey crushed stone with sand, trace brick and mortar (1.30)		SS	2	21	13	1	84.99		
FILL: Brown silty fine sand with crushed stone, gravel, cobbles, trace brick, mortar and clay		SS	3	33	15				
						2	83.99		
		SS	4	38	28				
(2.84)		SS	5		50+				
End of Borehole Practical spoon refusal at 2.84m depth									

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Perimeter Wall Rehabilitation - West Slope
Parliament Hill, Ottawa, Ontario

DATUM Borehole ground surface elevations referenced to a geodetic datum.

FILE NO. PG2779

REMARKS

HOLE NO. BH 3

BORINGS BY Portable Drill **DATE** October 26, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	85.50		
		AU	1						
FILL: Brown silty sand, some gravel, trace organics, clay, crushed stone, cobbles, concrete and mortar		SS	2	33	7	1	84.50		
		SS	3	33	24				
						2	83.50		
		SS	4	60	60				
2.44									
End of Borehole Practical spoon refusal at 2.44m depth									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Parliament Hill - West Slope
Ottawa, Ontario

DATUM TBM - Top of catchbasin (refer to Test Hole Location Plan). Geodetic elevation = 85.50m.

REMARKS

BORINGS BY Portable Drill

DATE October 13, 2011

FILE NO. PG2143

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	85.77		
TOPSOIL (0.05)									
FILL: Dark brown silty sand with gravel, cobbles and boulders		SS	1	8	5				
		SS	2	8	8	1	84.77		
		SS	3	0	50+				
		SS	4	33	10	2	83.77		
		SS	5	0	50+				
(3.35)						3	82.77		
BEDROCK: Grey limestone		RC	1	100	75				
(3.96) End of Borehole									

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Parliament Hill - West Slope
Ottawa, Ontario

DATUM TBM - Top of catchbasin (refer to Test Hole Location Plan). Geodetic elevation = 85.50m.

REMARKS

BORINGS BY Portable Drill

DATE October 14, 2011

FILE NO. PG2143

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	84.22		
TOPSOIL — 0.08									
FILL: Dark brown to black silty sand with gravel, cobbles and boulders		SS	1	100	6				
		SS	2	100	4	1	83.22		
		SS	3	4	3				
		SS	4	4	5	2	82.22		
— 2.62		SS	5	14	50+				
BEDROCK: Grey limestone		RC	1	60	50	3	81.22		
		RC	2	100	90				
— 4.01						4	80.22		
End of Borehole									

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Parliament Hill - West Slope
Ottawa, Ontario

DATUM TBM - Top of catchbasin (refer to Test Hole Location Plan). Geodetic elevation = 85.50m.

REMARKS

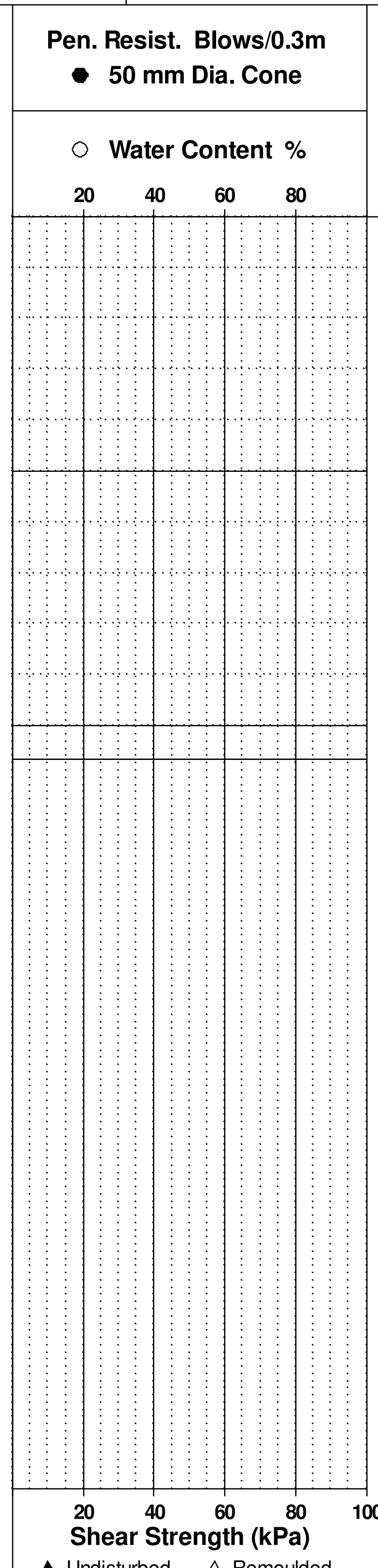
BORINGS BY Portable Drill

DATE October 14, 2011

FILE NO. PG2143

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	84.91
TOPSOIL 0.05							
FILL: Brown to black silty sand with gravel, cobbles and boulders		SS	1	12	7		
		RC	2	14	7	1	83.91
1.14							
BEDROCK: Grey limestone		RC	1	60	40	2	82.91
2.13							
End of Borehole							



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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

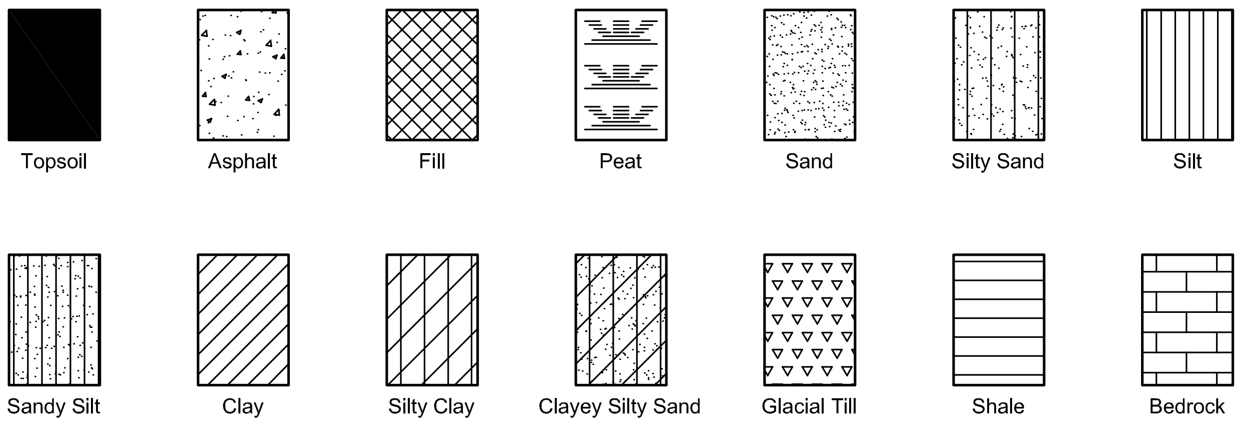
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

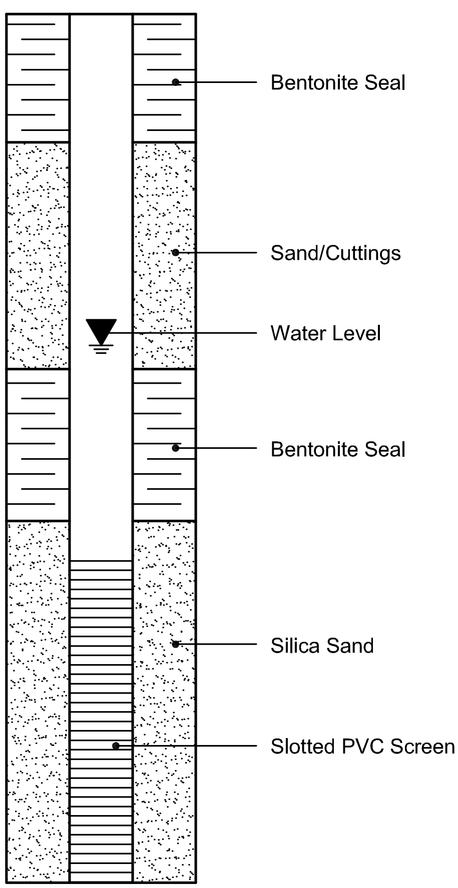
SYMBOLS AND TERMS (continued)

STRATA PLOT

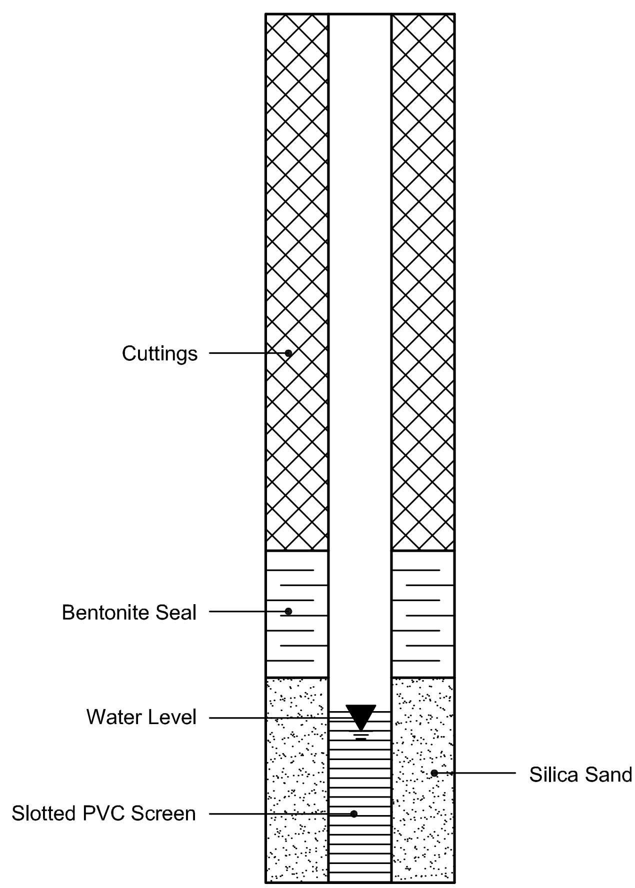


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



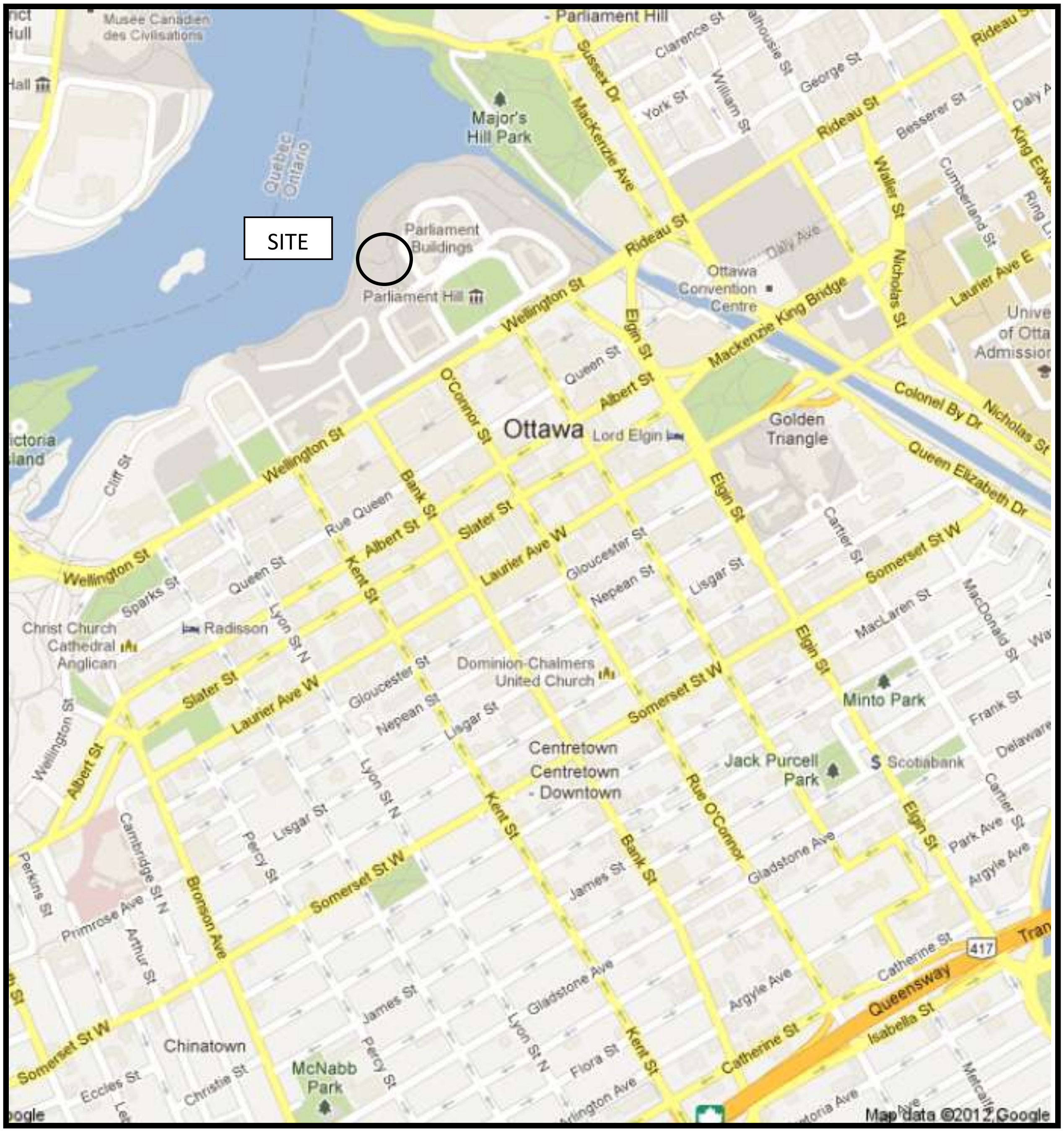
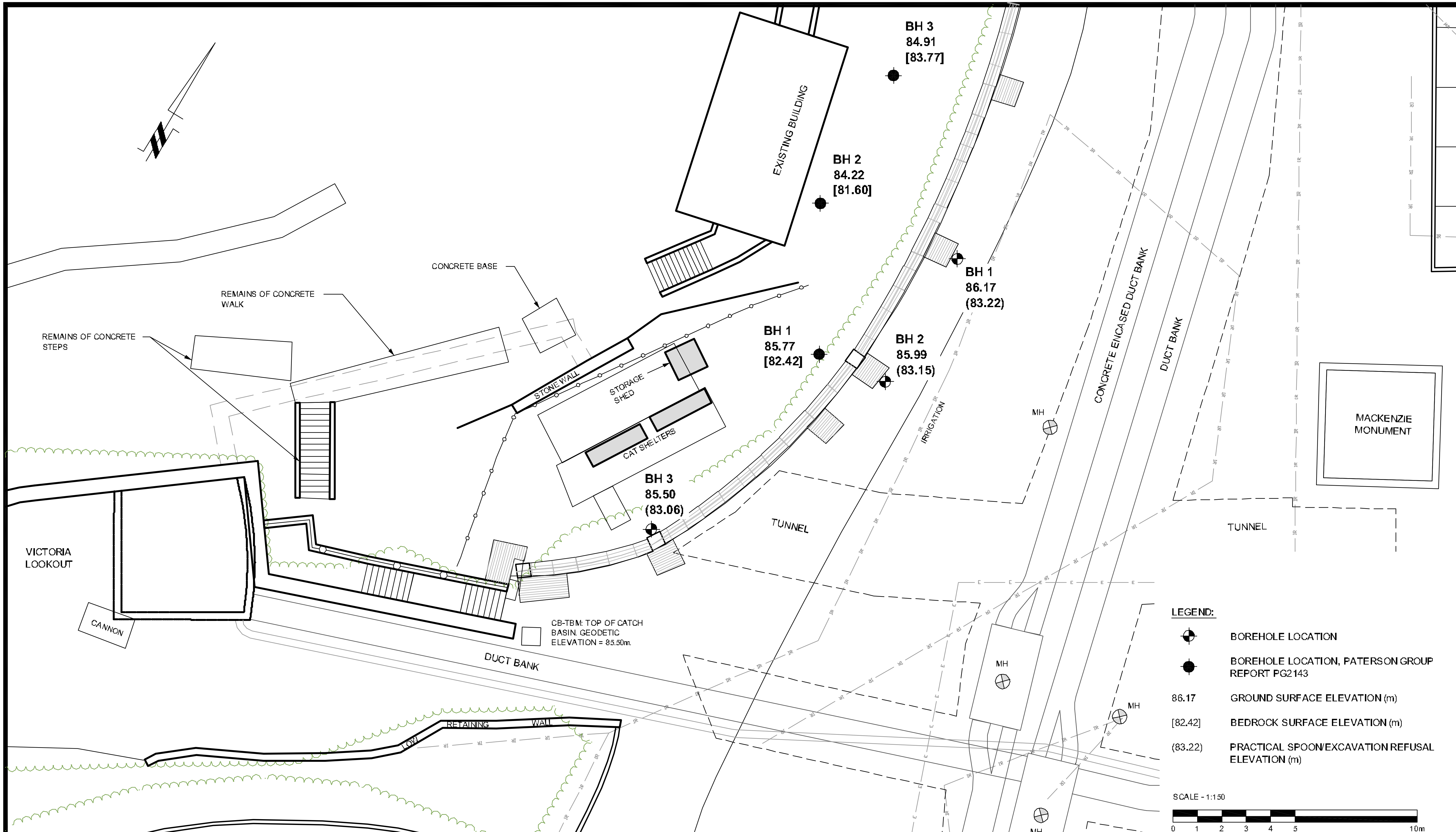


FIGURE 1

KEY PLAN

patersongroup



patersongroup consulting engineers 154 Colonnade Road South, Ottawa, Ontario K2E 7J5	Scale: 1:150	ROBERTSON MARTIN ARCHITECTS INC.	Dwg. No. PG2779-1
	Des.: RG	GEOTECHNICAL INVESTIGATION PROPOSED PERIMETER WALL REHABILITATION WEST SLOPE, PARLIAMENT HILL	TEST HOLE LOCATION PLAN
	Dwn: MPG		Report No. PG2779-1
	Chkd: DG	OTTAWA, ONTARIO	Date: 11/2012

11x17