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Geotechnical Engineering
Environmental Engineering
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May 29, 2012
File: PG2622-LET.02

Holzman Consultants
1076 Castle Hill Crescent
Ottawa, Ontario
K2C 2A8

Attention: **Mr. Bill Holzman**

Subject: **Preliminary Geotechnical Investigation**
Proposed Residential Building
1131 Teron Road - Ottawa

Dear Sir,

Further to your request, Paterson Group (Paterson) conducted a preliminary geotechnical investigation for the proposed development to be constructed at the aforementioned site.

Field Investigation

The field program for this investigation was conducted on May 17, 2012, and consisted of three (3) boreholes completed using a track-mounted drill rig operated by a local contractor. 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 elevation 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 top spindle of the fire hydrant located along the south east of Teron Road, adjacent to the subject site. The locations and ground surface elevations of the boreholes and the TBM are shown on Drawing PG2622-1 - Test Hole Location Plan attached to the present letter.

Field Observations

The subject site is currently divided into two (2) portions. The northern portion is undeveloped, grass covered with several small trees. A group of mature trees separates the northern portion of the site from the southern. The southern portion of the site consists of an existing building with parking areas.

The subsurface profile encountered at the test hole locations consisted of topsoil overlying a silty sand deposit. Silty clay was encountered below the silty sand deposit. Practical refusal to the dynamic cone penetration test (DCPT) was noted at 16.4 m in BH 2. 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 pit locations.

Based on available geological mapping, the bedrock in the immediate area consists of migmatic rocks and paragneiss with an overburden thickness of 15 to 25 m depth.

Groundwater levels were recorded from piezometers installed at the borehole locations of the present investigation. The groundwater readings at the borehole locations are presented in Table 1. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Borehole Number	Ground Surface Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BH 1	89.53	1.11	88.42	May 23, 2012
BH 2	89.38	1.15	88.23	May 23, 2012
BH 3	89.35	1.05	88.30	May 23, 2012

Note: The ground surface elevations at the borehole locations were referenced to a TBM, consisting of the top spindle of the fire hydrant located along the south side of Bridgestone Drive, adjacent to the subject site. An geodetic elevation of 91.265 m was provided for the TBM.

Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed building. It is expected that the building will be founded on conventional spread footings, end bearing piles or on a raft foundation.

Due to the presence of a deep silty clay deposit, a permissible grade raise restriction of 1.5 m is recommended for the subject site

Special attention should be given to the prevention of groundwater lowering within the vicinity of the proposed development. The effect of groundwater lowering could result in settlement of the neighbouring buildings. Consideration may be given to waterproofing the lower levels of the parking garage.

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

Site Grading and Preparation

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

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

Foundation Design

Shallow Foundation

For the garage portion extending beyond the building foundation, and provided the building loads can accommodate a spread footing design, the proposed building could be constructed on shallow footings.

A bearing resistance value at serviceability limit states (SLS) of **75 kPa** and a factored bearing resistance value at ultimate limit states (ULS) **125 kPa** can be used for pad or column footings, up to 6 m, square or and strip footings, up to 4 m wide, placed on the firm silty clay deposit. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS. The bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 15 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. Above the groundwater level, adequate lateral support is provided to a firm to very stiff silty clay to clayey silt, dense glacial till or engineered fill 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.

Raft Foundation

Consideration can be given to a raft foundation to found the proposed multi-storey building. The factored bearing resistance (contact pressure) at ultimate limit states (ULS) can be taken to be **200 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The design should allow for the use of no more than 80% of the overconsolidation of the silty clay and account for a potential 0.5 m post-development groundwater lowering. A bearing resistance value at serviceability limit states (SLS) (contact pressure) of **125 kPa** can be used. 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 modulus of subgrade reaction was calculated to be **3.3 MPa/m** for a contact pressure of **125 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modeling the soil structure interaction is to consider the bearing medium to be elastic and to assign a subgrade modulus. However, sensitive silty clay is not elastic and limits have to be placed on the stress ranges of a particular modulus. This value can be re-evaluated once detail of the structural design becomes available.

Based on the above assumptions for the raft foundation, the proposed building can be designed using the above parameters and a total and differential settlement of 25 and 15 mm, respectively. The base of the raft foundation (approximately 1 to 1.5 m thick) is to be located at a depth of approximately 8 to 10 m below the original ground surface.

Deep Foundation

For support of the proposed multi-storey building consideration could be given to using concrete filled steel pipe piles driven to refusal on the bedrock surface.

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values 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. For this project, the dynamic monitoring of two (2) to four (4) piles would be recommended. As a minimum, the pipe piles should be equipped with a base plate having a thickness of at least 20 mm to minimize damage to the pile tip during driving. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	940	1130	10	29
245	11	1175	1410	10	35
245	13	1375	1650	10	42

Design for Earthquakes

The proposed building can be designed using a seismic site response **Class D** as defined in the Ontario Building Code 2006 (OBC 2006; Table 4.1.8.4.A) for shallow foundations considered at this site. The soils underlying the site are not susceptible to liquefaction.

Basement Slab

It is expected that the basement area will be mostly parking and that a concrete slab with a subfloor granular layer will be incorporated in the design to accommodate services and a rigid pavement structure noted in the below section will be applicable. The thickness of the granular subfloor layer will be dependent on what services are incorporated in the design. It is also expected that a sump pit will be incorporated to drain any water which enters the granular layer via a breach in the raft slab.

A concrete mud slab will be used to protect the native soil from worker traffic and equipment before pouring the raft slab. A waterproofing membrane placed between the raft slab and the mud slab will only be considered if the groundwater infiltration is excessive. It is expected that the diaphragm walls will significantly reduce water infiltration volumes.

Pavement Structure

Asphalt pavement for above grade parking will be required for the subject site. The recommended flexible pavement structures shown in Tables 3 and 4 would be applicable.

Table 3 - Recommended Flexible 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 4 - Recommended Flexible Pavement Structure - Truck and Access Lanes

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
400	**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 98% of the material's SPMDD using suitable vibratory equipment.

Protection of Footings Against Frost Action

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

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

Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems 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 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

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.

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 Holzman Consultants or their agents, without review by this firm for the applicability of our recommendations to the altered use of the report.

Best Regards,

Paterson Group Inc.

Stephanie Boisvenue, B.Eng.

Carlos P. Da Silva, P.Eng.

Attachments

- ❑ Soil Profile and Test Data sheets
- ❑ Figure 1 - Key Plan
- ❑ Drawing PG2622-1 - Test Hole Location Plan

Report Distribution

- ❑ Holzman Consultants (3 copies)
- ❑ Paterson Group (1 copy)

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - 1131 Teron Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of subject site on Teron Road. Geodetic elevation = 91.265m.

REMARKS

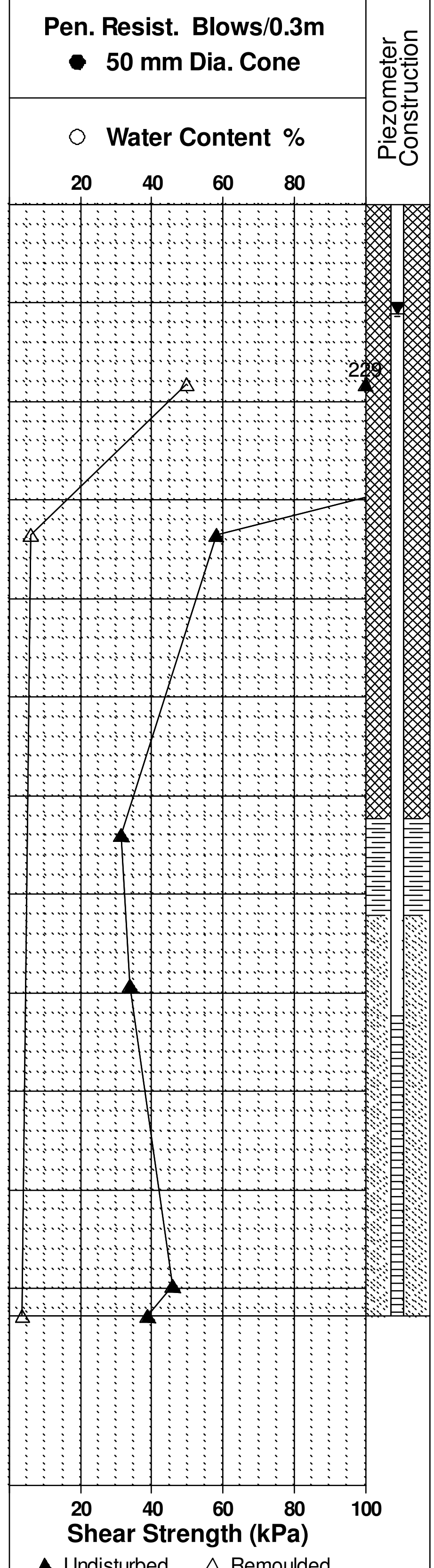
BORINGS BY CME 55 Power Auger

DATE May 17, 2012

FILE NO. PG2622

HOLE NO. BH 1

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.53
TOPSOIL	0.25	AU	1				
Brown SILTY SAND	0.69	AU	2				
Hard to very stiff, brown SILTY CLAY		SS	3	83	9	1	88.53
- stiff to firm and grey by 2.4m depth						2	87.53
						3	86.53
						4	85.53
						5	84.53
						6	83.53
						7	82.53
						8	81.53
						9	80.53
						10	79.53
	11.28					11	78.53
End of Borehole							
(GWL @ 1.11m-May 23, 2012)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - 1131 Teron Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of subject site on Teron Road. Geodetic elevation = 91.265m.

REMARKS

BORINGS BY CME 55 Power Auger **DATE** May 17, 2012

FILE NO. PG2622

HOLE NO. BH 2

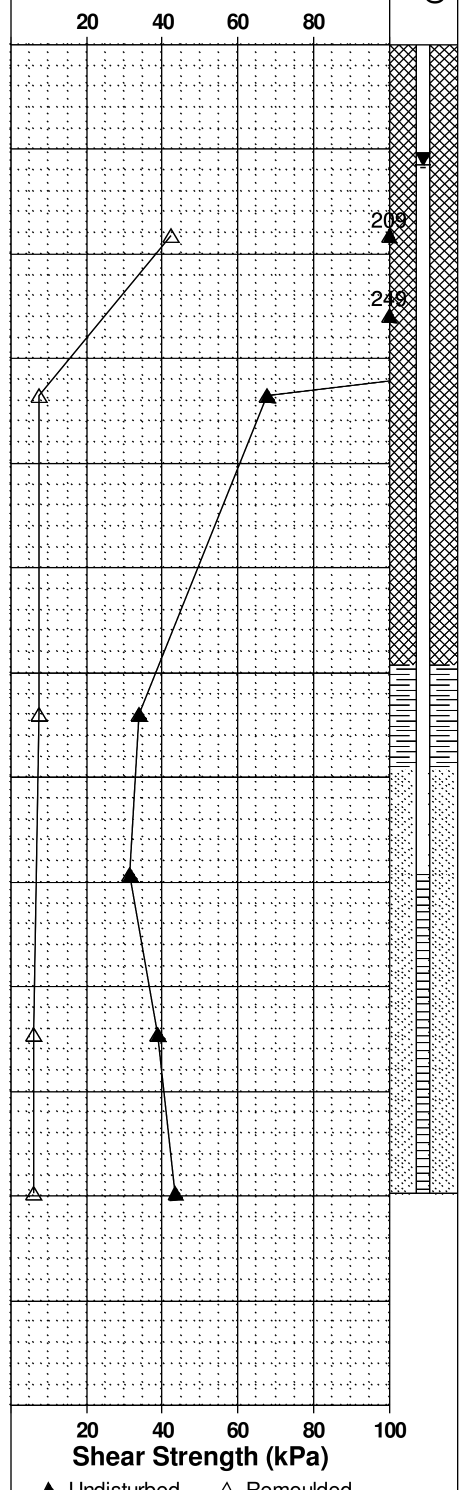
SOIL DESCRIPTION	Layer bottom depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.23				
Brown **SILTY SAND**	0.60	AU	1		
Brown **SILTY CLAY** with sand seams	1.45	SS	2	83	9
Hard to very stiff, brown **SILTY CLAY** - stiff to firm and grey by 2.8m depth	10.97				
Dynamic Cone Penetration Test commenced at 10.97m depth. Cone pushed to 16.2m depth.					

DEPTH (m)	ELEV. (m)
0	89.38
1	88.38
2	87.38
3	86.38
4	85.38
5	84.38
6	83.38
7	82.38
8	81.38
9	80.38
10	79.38
11	78.38
12	77.38
13	76.38

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

○ Water Content %

Piezometer Construction



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - 1131 Teron Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of subject site on Teron Road. Geodetic elevation = 91.265m.

REMARKS

BORINGS BY CME 55 Power Auger **DATE** May 17, 2012

FILE NO. PG2622

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 %	Piezometer Construction
GROUND SURFACE						13	76.38		
						14	75.38		
						15	74.38		
						16	73.38		
16.43									
End of Borehole Practical DCPT refusal at 16.43m depth (GWL @ 1.15m-May 23, 2012)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Building - 1131 Teron Road
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant located in front of subject site on Teron Road. Geodetic elevation = 91.265m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE May 17, 2012

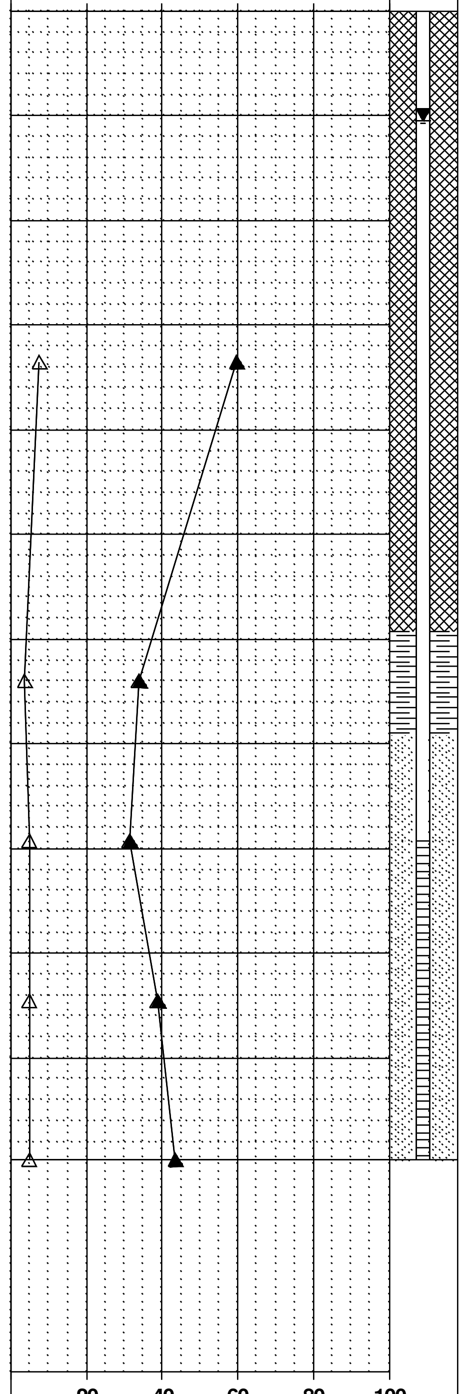
FILE NO. PG2622

HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	89.35
TOPSOIL (0.25)		AU	1				
Brown SILTY SAND		SS	2	83	13	1	88.35
(1.45)		SS	3		7		
Very stiff to stiff, brown SILTY CLAY						2	87.35
- stiff to firm and grey by 2.6m depth						3	86.35
						4	85.35
						5	84.35
						6	83.35
						7	82.35
						8	81.35
						9	80.35
						10	79.35
(10.97) End of Borehole							
(GWL @ 1.05m-May 23, 2012)							

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

○ Water Content %



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 = $(D30)^2 / (D10 \times D60)$
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

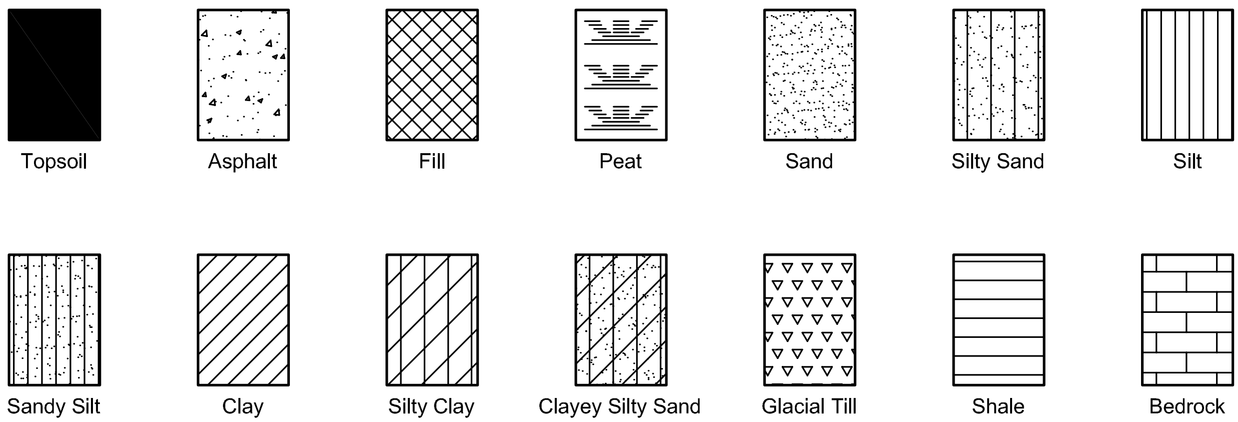
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

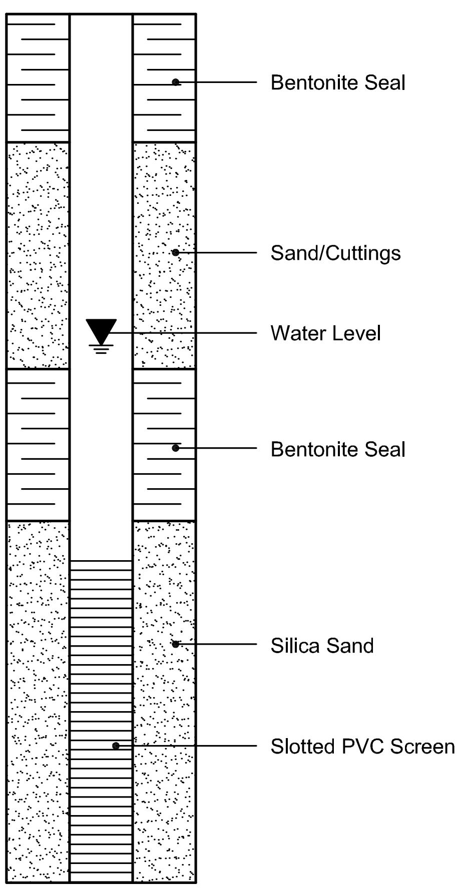
SYMBOLS AND TERMS (continued)

STRATA PLOT

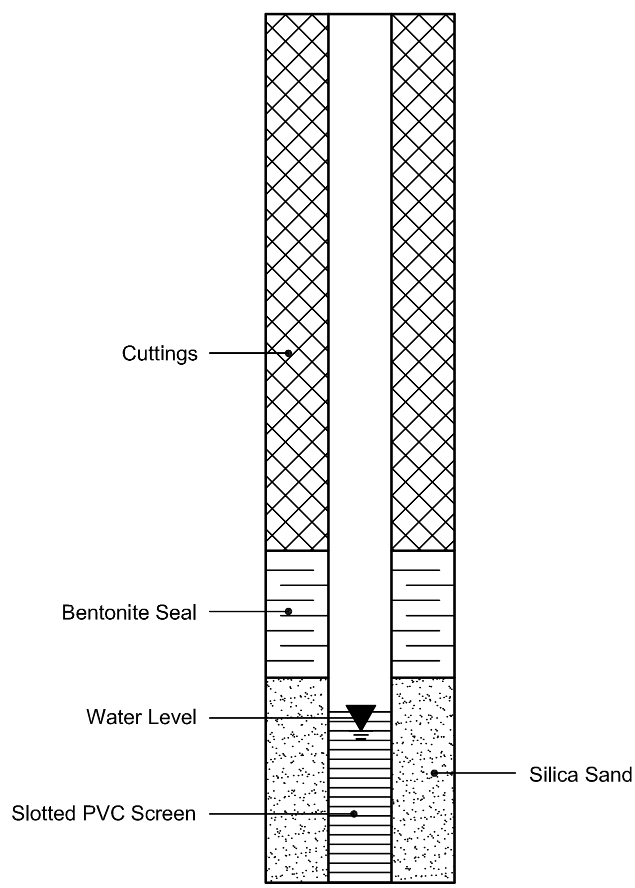


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



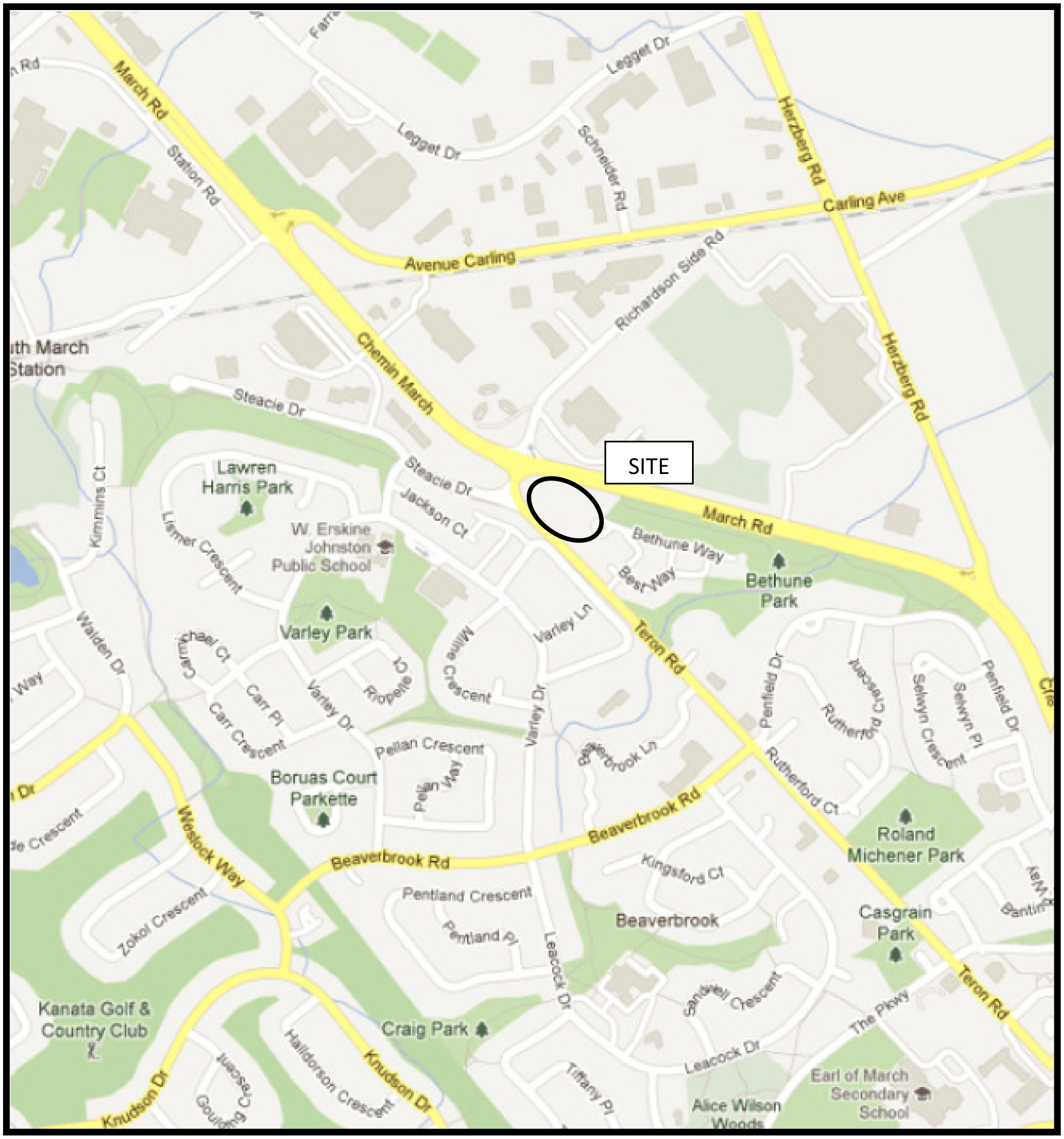
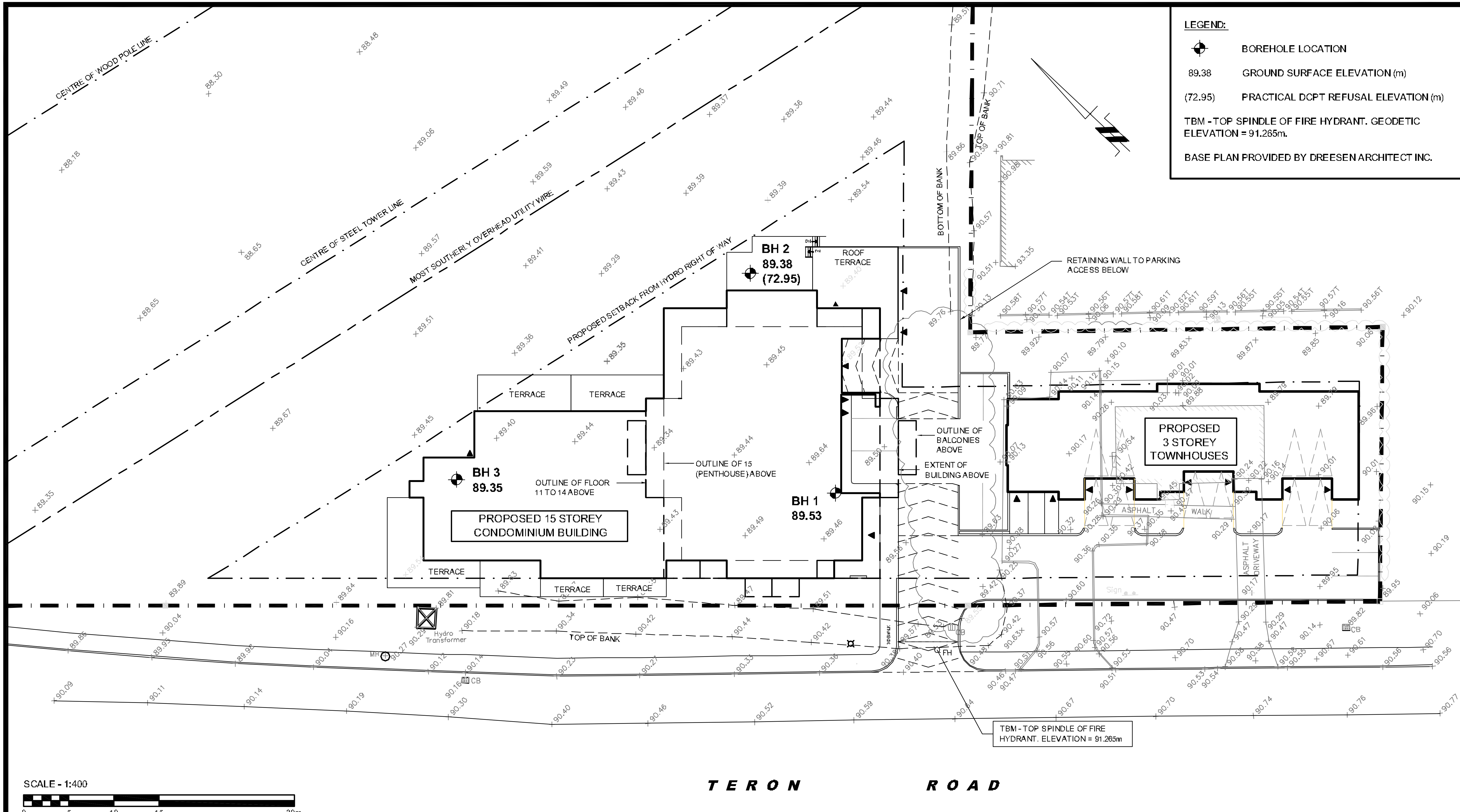


FIGURE 1

KEY PLAN



paterson group consulting engineers 154 Colonnade Road South, Ottawa, Ontario K2E 7J5	Scale: 1:400 Des.: SB Dwn: MPG Chkd: CDS	HOLZMAN CONSULTANTS INC. GEOTECHNICAL INVESTIGATION PROP. RESIDENTIAL DEVELOPMENT-1131 TERON RD. OTTAWA, ONTARIO	**TEST HOLE LOCATION PLAN**	Dwg. No. **PG2622-1** Report No.: PG2622-1 Date: 05/2012

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