Geotechnical Investigation for a Commercial Property Located at 14027 Hurontario Street, Inglewood, Ontario

Report # 4545 – BVD Caledon
October 25, 2019

Prepared for:
Bikram Dhillon
BVD Petroleum
Tel: 416-848-4111
E-mail: bikram.dhillon@bvdpetroleum.com

Prepared by:
A &A Environmental Consultants Inc.
16 Young Street
Woodstock, ON N4S 3L4
Tel: 519-266-4680
Fax: 519-266-3666
www.aaenvironmental.ca

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1.0 INTRODUCTION

1.1 Proposed Construction

Bikram Dhillon of BVD Petroleum (the Client), retained the services of A & A Environmental Consultants Inc. (A&A) to conduct a geotechnical investigation for a proposed commercial development on a property located at 14027 Hurontario Street, Caledon, Ontario. The site is located on the northeast corner of the intersection of Hurontario Street (Highway 10) and King Street. Eight boreholes were to be advanced and sampled for this geotechnical investigation. The information obtained is to provide recommendations that will allow the design of foundations and pavement at the site. See Section 4 for additional details of the proposed development.

1.2 Purpose and Limitations of Report

The purpose of this study is to provide geotechnical information, recommendations and comments for the design and construction of the proposed development. The number of boreholes has been selected to provide representative information sufficient to determine parameters needed for design, specifications and construction of the proposed development. Conditions elsewhere near or beneath the footprint of the structures may be found to differ, during construction, from those at the borehole locations. Should this occur, the contractor should contact the design engineer for recommendations as how to best proceed and what changes if any, should be made.

The information in this report is intended for this specific proposed structure and has been prepared for the client, and their nominated engineers and designers. It is assumed that the designers will use all appropriate contemporary standards, governing regulations, and codes in the performance of their work. Third party use or reproduction, in part or in full, of this report is prohibited without written authorization from A&A. This report is also subject to the Statement of Limitations which from an integral part of this document.

1.3 Liaison during design and/or Construction

On-going liaison with A&A during the final design and construction phases of the project is recommended to confirm that they are in keeping with the intentions of this report.

2.0 SCOPE OF WORK

2.1 Proposed Scope of Work

The scope of work for the geotechnical investigation of the proposed development is as follows:

- Advance eight boreholes to sample for geotechnical analysis. All eight boreholes will be advanced to a maximum depth of 9.14 meters below ground level(mbgl) (30 feet).
- Submit select soil samples to a geotechnical laboratory to provide information for the soil samples recovered.
- Prepare a geotechnical report summarizing the results of the field investigation and laboratory testing program, to include discussion of specific concerns that need to be addressed during design and/or construction. Specifically, the report is to include:
    - Site plan showing locations of the boreholes;
    - Borehole records;
    - Recommendations for:
        - Site preparation;
        - Construction dewatering if required;
        - Earthworks;
        - Potential reuse of existing fill materials and/or native soils indicated in the boreholes;
        - Excavation requirements;
        - Geotechnical bearing reactions and resistances for foundation designs;
        - Lateral earth pressure coefficients for existing soils and typical imported materials;

3.0 SITE DESCRIPTION

3.1 Current Land Use and Location

The site is zoned as being, "CV-267 – Village Commercial" as quoted from the Town of Caledon By-law 2006-50 and is located at 14027 Hurontario Street (Figure 1, Appendix A). The approximate UTM coordinates of the site are Zone 17T; 590279m Easting and 4847461m Northing. The area inspected is rectangular in shape and is currently comprised of a vacant single storey residence and an overgrown field.

3.2 Topography and Drainage

The area around the subject site has a slope south/southeastward towards a tributary of Etobicoke Creek. According to the topographic map acquired from Natural Resources Canada website the elevation of the subject site is approximately 287 meters above sea level (masl) (Figure 2, Appendix A).

The surface deposit in this region, like all of Ontario, was once covered by massive glaciers during the late Wisconsin glacial period. The grinding action of the moving ice masses produced a considerable amount of rock materials, ranging in size from boulders to rock flour which was distributed over the landscape. The Ministry of Northern Development Mines and Forestry offers a feature for Google Earth TM that maps various geological types for Ontario:

- The “Bedrock Geology of Ontario” identifies the site within the Queenston Formation characterized by shale, limestone, dolostone, siltstone.
- The “Paleozoic Geology of Southern Ontario” identifies the site to be within the Queenston Formation characterized by shale, siltstone, minor limestone and sandstone.
- The “Physiography of Southern Ontario” identifies the site to be part of the Till Plains (Drumlinized) formation within the South Slope region.
- The “Quaternary Geology” identifies the site as Halton Till characterized by predominantly silt to silty clay matrix, high in matrix carbonate content and clast poor.
- The “Surficial Geology” identifies the site as Till characterized by Clay to silt-textured till (derived from glaciolacustrine deposits or shale).

4.0 PROPOSED DEVELOPMENT

It is understood that the proposed RFO will consist of the following:

- One building having a footprint of 600 m^2 consisting of a convenience store and a restaurant with a drive thru;
- One building having a footprint of 293 m^2 consisting of a Burger King restaurant and a drive thru;
- Two future buildings having a footprint of 332 m^2;
- Five underground storage tanks (USTs);
- Four fuel pump islands with a canopy;
- Five fuel pump cardlock island;
- A parking area for:
  - 112 total parking spaces, which includes 5 spaces for transport trucks;
- There will be two access points, one from Hurontario Street in the northwest corner of the site and the other from King Street in the southwest corner of the site.

The general arrangement of the proposed development is illustrated in Figure 3, Appendix A.

5.0 METHOD OF INVESTIGATION

5.1 Field Investigation

A&A engaged a utility locating company to map locations of public and private underground utilities. A&A then scheduled the drilling of boreholes for sampling in accordance with the borehole drilling and sampling plan.

The geotechnical investigation for the planned development consisted of the following activities:

- On July 12, 2019 and October 1, 2019, A&A attended the site located at 14027 Hurontario Street, Caledon, Ontario.
- Boreholes were advanced using a track mounted drill unit with hollow stem augers. Split spoon samplers were used for standard penetration tests and to obtain soil samples from the boreholes. The stratigraphy in each borehole was recorded in the field at regular intervals and samples collected by the A&A personnel. Standard Penetration Tests (SPTs), followed the methods described in ASTM Standard D1586-08a. The blows were recorded for a depth of 460 mm (18”) as shown on the borehole logs. The last two 150 mm distances (total = 300 mm) are used to calculate the *N* value. See Table 1 for each borehole advanced depth and location. Figure 4 in Appendix A depicts the locations of the boreholes in relation to the proposed development. Samples submitted for analysis are to be representative of the boreholes and their location within the proposed development.

Table 1 – Borehole Advanced Depths and Location

Borehole	Location	Depth (ft.)/(m)
BH-1	North section of the subject site, within the footprint of the retail fuel pump island, canopy, and USTs	20/6.1
BH-2	Middle of the subject site, within the footprint of the commercial fuel pump island, canopy, and USTs	30/9.1
BH-3	South area of the subject site, within the footprint of the proposed future development	23/7.0
BH-4	Mid-west area of the subject site, within the footprint of the proposed Burger King restaurant building	10/3.0
BH-5	West area of the subject site, within the footprint of the proposed parking area	10/3.0
BH-6	Mid-west area of the subject site, within the footprint of the proposed C-Store and restaurant building	10/3.0
BH-7	Northwest corner area of the subject site, within the footprint of the proposed right-turn lane off of Hurontario Street	10/3.0
BH-8	South-middle area of the subject site, within the footprint of the proposed right-turn lane off of King Street.	10/3.0

- All boreholes were used for geotechnical sampling. All geotechnical boreholes except BH-1, BH-2, and BH-3 were refilled with low-permeability bentonite pellets. BH-1, BH-2, and BH-3 had a monitoring well installed during the investigation. These wells will be used mainly for the hydrogeological investigation but will be used for the water table elevation for this investigation.

5.2 Sampling Procedures

Select samples recovered from the geotechnical investigation were submitted to Terraprobe Inc. (Terraprobe), a certified geotechnical and materials testing laboratory. The scope of the geotechnical laboratory testing program includes the following:

- In-situ water content per ASTM D2216;
- Grain size analyses per ASTM D422 & D2217;
- Atterburg Limits per ASTM 4318;

The results of the laboratory tests are discussed in the text of this report. The results of the moisture content tests are shown on the Borehole Records in Appendix B. The results of the grain size distribution tests are also shown on the borehole records (Appendix B) and are illustrated in Appendix C.

6.0 LABORATORY TESTING AND RESULTS OF INVESTIGATION

6.1 Subsurface Conditions Overview

The borehole logs provided in Appendix B summarize the soil types observed during drilling. Explanation of the symbols and terms used to describe the borehole records are also included in Appendix B.

Select bagged samples taken from the boreholes were analyzed at Terraprobe for natural moisture content, grain size analysis and Atterberg limits.

It should be noted that the boundaries between the strata on the borehole records have been inferred from drilling observations and non-continuous sampling. The boundaries generally represent a transition from one soil type to another and should not be inferred to represent an exact plane of geological change. Further, conditions will vary between and beyond the boreholes.

The subsurface conditions encountered in the boreholes typically consist of a thin surface layer of asphalt. This is followed by a layer of sand and gravel, which extends to 0.76 mbgl. Beyond 0.76 mbgl, there is a layer of clayey silt, some sand, trace gravel. It should be noted that the soil of this composition will behave geotechnically much like a clay soil. BH1 through BH8 were terminated within the clay layer at depths ranging from 2.7 to 4.6 mbgl.

All boreholes were terminated due to either refusal or being below the depth of the intended foundation of each borehole location in reference to the site plan.

The drilling program for this study indicates that the overburden deposits are consistent across boreholes at the approximate proposed foundations depth of 1.2 m to 2.5 m. BH-1 to BH-8 has clayey silt some sand trace gravel mixture at the foundation depth. The strength variations are detailed in the borehole logs in Appendix B.

The combination of lab results and standard penetration test N values (blows/foot) were then used to estimate allowable bearing and shear values. This translation was based on generally accepted, recorded correlations from thousands of similar tests. Soil characteristics for each hole may be found in Appendices B & C.

6.2 Detailed Summary

After the topsoil material, a layer of clayey silt, some sand, trace gravel spreads across the site starting at a depth between 1.0-1.5 mbgl and continuing to 4.0-5.0 mbgl. This layer was brown to greyish in colour. Water content increased with depth with a saturation point at approximately 3.05 mbgl. There was an average of 21 – 37 blows/foot within this layer which is considered stiff to very stiff. All boreholes were terminated within this deposit.

Atterberg limits testing was carried out on all eight samples of this deposit and measured plasticity for all the samples. The liquid limit varied from 28.44% to 33.01%. The plastic limit varied from 14.71% to 16.91%. The plasticity index varied from 13.48% to 17.02%. The water content varied from 7.4% to 17.3%. Based on the results, all of the boreholes (BH1-8) are considered clay with "low plasticity" in nature.

6.2.1 Summary of BH-1

The soil in borehole BH-1 is consistent with the layers described above. The SPT-N values at various depths are as shown on the borehole logs and are described in Table 2 below. The geotechnical SPT's did not continue past 3.51 mbgl due to being below the depth of the intended foundation.

Table 2 – Blow Counts of BH-1

Depth (mbgl)	STP – N Depth (mbgl)	SPT – N (Blows/0.3 m)
0.0 to 0.46	0.3	17
0.76 to 1.22	1.05	7
1.52 to 1.98	1.8	21
2.29 to 2.74	2.55	37
3.05 to 3.51	3.3	55

6.2.2 Summary of BH-2

The soil in borehole BH-2 is consistent with the layers described above. The SPT-N values at various depths are as shown on the borehole logs and are described in Table 3 below. The geotechnical SPT's did not continue past 4.27 mbgl due to refusal.

Table 3 – Blow Counts of BH-2

Depth (mbgl)	STP – N Depth (mbgl)	SPT – N (Blows/0.3 m)
0.0 to 0.46	0.3	2
0.76 to 1.22	1.05	9
1.52 to 1.98	1.8	9
2.29 to 2.74	2.55	29
3.05 to 3.51	3.3	50
3.81 to 4. 27	4.1	>50

6.2.3 Summary of BH-3

The soil in borehole BH-3 is consistent with the layers described above. The SPT-N values at various depths are as shown on the borehole logs and are described in Table 4 below. The geotechnical SPT's did not continue past 4.27 mbgl due to refusal.

Table 4 – Blow Counts of BH-3

Depth (mbgl)	STP – N Depth (mbgl)	SPT – N (Blows/0.3 m)
0.0 to 0.46	0.3	22
0.76 to 1.22	1.05	7
1.52 to 1.98	1.8	28
2.29 to 2.74	2.55	44
3.05 to 3.51	3.3	50
3.81 to 4. 27	4.1	>50

6.2.4 Summary of BH-4

The soil in borehole BH-4 is consistent with the layers described above. The SPT-N values at various depths are as shown on the borehole logs and are described in Table 5 below. The geotechnical SPT's did not continue past 2.74 mbgl due to being below the depth of the intended foundation.

Table 5 – Blow Counts of BH-4

Depth (mbgl)	STP – N Depth (mbgl)	SPT – N (Blows/0.3 m)
0.0 to 0.46	0.3	10
0.76 to 1.22	1.05	8
1.52 to 1.98	1.8	26
2.29 to 2.74	2.55	45

6.2.5 Summary of BH-5

The soil in borehole BH-5 is consistent with the layers described above. The SPT-N values at various depths are as shown on the borehole logs and are described in Table 6 below. The geotechnical SPT's did not continue past 2.74 mbgl due to being below the depth of the intended foundation.

Table 6 – Blow Counts of BH-5

Depth (mbgl)	STP – N Depth (mbgl)	SPT – N (Blows/0.3 m)
0.0 to 0.46	0.3	11
0.76 to 1.22	1.05	22
1.52 to 1.98	1.8	31
2.29 to 2.74	2.55	38

6.2.6 Summary of BH-6

The soil in borehole BH-6 is consistent with the layers described above. The SPT-N values at various depths are as shown on the borehole logs and are described in Table 7 below. The geotechnical SPT's did not continue past 2.74 mbgl due to being below the depth of the intended foundation.

Table 7 – Blow Counts of BH-6

Depth (mbgl)	STP – N Depth (mbgl)	SPT – N (Blows/0.3 m)
0.0 to 0.46	0.3	8
0.76 to 1.22	1.05	10
1.52 to 1.98	1.8	26
2.29 to 2.74	2.55	34

6.2.7 Summary of BH-7

The soil in borehole BH-7 is consistent with the layers described above. The SPT-N values at various depths are as shown on the borehole logs and are described in Table 8 below. The geotechnical SPT's did not continue past 2.74 mbgl due to being below the depth of the intended foundation.

Table 8 – Blow Counts of BH-7

Depth (mbgl)	STP – N Depth (mbgl)	SPT – N (Blows/0.3 m)
0.0 to 0.46	0.3	61
0.76 to 1.22	1.05	32
1.52 to 1.98	1.8	31
2.29 to 2.74	2.55	10

6.2.8 Summary of BH-8

The soil in borehole BH-8 is consistent with the layers described above. The SPT-N values at various depths are as shown on the borehole logs and are described in Table 9 below. The geotechnical SPT's did not continue past 2.74 mbgl due to being below the depth of the intended foundation.

Table 9 – Blow Counts of BH-8

Depth (mbgl)	STP – N Depth (mbgl)	SPT – N (Blows/0.3 m)
0.0 to 0.46	0.3	29
0.76 to 1.22	1.05	0
1.52 to 1.98	1.8	14
2.29 to 2.74	2.55	37

6.3 Groundwater Conditions

Groundwater and surface water are expected to flow towards the natural slope of the ground surface. Although the surface topography typically has great influence on the groundwater flow, it has been observed in several areas that bedrock topography also has a significant influence on the flow, in some cases more so than surface topography. In the latter case, this is believed to be due to relatively impermeable bedrock underlying a much more permeable silt overburden. Based on the topography, the surface water drainage and the regional scale mapping, groundwater flow in the overburden is inferred to be in an eastern direction, towards Sawmill Creek. Groundwater flow direction may also be influenced by utility trenches or other subsurface structures and may preferentially migrate in these subsurface utility trenches. The site is mostly flat with slight sloping towards the centre of the subject site. The water table location was inferred to be 1.4 – 1.9 mbgl according to Ontario well records for the site.

6.4 Summary of Subsurface Conditions to Anticipated Depths of Construction

In the following tables (Tables 10-12), the relevant properties of the various deposits are briefly described. For details of the subsurface conditions, reference should be made to the individual borehole logs.

It should be noted that the soil boundaries indicated on the borehole logs are obtained from non-continuous sampling and observations during drilling. These boundaries reflect transition zones, for the purpose of geotechnical design, and should not be interpreted as exact planes of geological change. The "Notes on Sample Description" preceding the borehole logs are an integral part of and should be read in conjunction with this report.

Table 10 – Typical Values of Moisture Content

Sample #	Depth ft(m)	Soil Description	Water Content (%)
BH1	10 – 12' (3.05 – 3.66)	Silty Clay some Sand and Gravel	NT
BH2	12.5 – 14.5' (3.8 – 4.4)	Clayey Silt some Sand trace Gravel	NT
BH3	5 – 7' (1.5 – 2.13)	Clayey Silt some Sand trace Gravel	NT
BH4	7.5 – 9.5' (2.29 – 2.9)	Clayey Silt some Sand trace Gravel	NT
BH5	2.5 – 4.5' (0.8 – 1.4)	Silt and Clay some Sand trace Gravel	NT
BH6	7.5 – 9.5' (2.29 – 2.9)	Silt and Clay some Sand trace Gravel	NT
BH7	2.5 – 4.5' (0.8 – 1.4)	Sand and Gravel trace Silt and Clay	7.4
BH8	7.5 – 9.5' (2.29 – 2.9)	Sandy Clay with Silt some Gravel	17.3

NOTE: NT- Not Tested

Table 11 – Typical Values of Atterburg Limits (%)

Sample #	Depth ft(m)	Soil Description	Atterburg Limits		
			W_L	W_P	I_P
BH1	10 – 12' (3.05 – 3.66)	Silty Clay some Sand and Gravel	28.75	15.11	13.64
BH2	12.5 – 14.5' (3.8 – 4.4)	Clayey Silt some Sand trace Gravel	30.50	15.84	14.66
BH3	5 – 7' (1.5 – 2.13)	Clayey Silt some Sand trace Gravel	30.39	16.91	13.48
BH4	7.5 – 9.5' (2.29 – 2.9)	Clayey Silt some Sand trace Gravel	29.75	15.38	14.37
BH5	2.5 – 4.5' (0.8 – 1.4)	Silt and Clay some Sand trace Gravel	33.01	15.99	17.02
BH6	7.5 – 9.5' (2.29 – 2.9)	Silt and Clay some Sand trace Gravel	29.74	16.19	13.55
BH7	2.5 – 4.5’ (0.8 – 1.4)	Sand and Gravel trace Silt and Clay	-	Non Plastic	-
BH8	7.5 – 9.5’ (2.29 – 2.9)	Sandy Clay with Silt some Gravel	28.44	14.71	13.74

Table 12 – Sieve and Hydrometer Analysis

BH #	Grain Size Content (%)				Sample Depth	Sample Description
	Gravel	Sand	Silt	Clay	Ft (m)	
BH1	10.9	18.7	34.5	35.8	10 – 12' (3.05 – 3.66)	Silty Clay some Sand and Gravel
BH2	6.8	20.7	38.7	33.8	12.5 – 14.5' (3.8 – 4.4)	Clayey Silt some Sand trace Gravel
BH3	8.4	22.2	37.5	31.9	5 – 7' (1.5 – 2.13)	Clayey Silt some Sand trace Gravel
BH4	6.5	21.7	29.8	32.0	7.5 – 9.5' (2.29 – 2.9)	Clayey Silt some Sand trace Gravel
BH5	8.8	19.0	37.1	35.1	2.5 – 4.5' (0.8 – 1.4)	Silt and Clay some Sand trace Gravel
BH6	4.8	20.5	36.8	38.0	7.5 – 9.5' (2.29 – 2.9)	Silt and Clay some Sand trace Gravel
BH7	39.3	48.9	8.4	3.4	2.5 – 4.5’ (0.8 – 1.4)	Sand and Gravel trace Silt and Clay
BH8	10.6	28.8	27.3	33.4	7.5 – 9.5’ (2.29 – 2.9)	Sandy Clay with Silt some Gravel

6.5 Resistance at Ultimate and Serviceability Limit States

According to current foundation engineering practice in Ontario, Limit States Design (LSD) approach is employed to estimate geotechnical resistance of shallow and/or deep foundations at ultimate limit states (ULS) and serviceability limit states (SLS). The recommended values presented in this report should be used by a structural engineer for the foundation design.

The groundwater table (GWT) is located approximately 1.4 – 1.9 mbgl. The saturated soil is susceptible to freezing that could lead to thawing and loss of soil strength. Thus, the footings should be founded at a minimum depth of 1.2 mbgl to minimize frost heave effect.

An average SPT-N value is calculated within the influence zone of the foundation, that is about one-half the footing width, B, above the estimated base location and $2B$ below. For the proposed development of a single storey concrete block building, the average SPT-N value across the boreholes within the approximate influence zone of 0.8 – 3.3 mbgl is:

$$N_{av.} = \frac{\sum N \cdot z_i}{\sum z_i}$$

The calculated average SPT-N value is deemed as a representative resistance at an approximate depth of $z = 2.3$ mbgl. To calculate the effective overburden pressure at the representative depth, a unit weight of 17 kN/m^3 is assumed for the soil based on the soil type and borehole observations. An average cohesion value of clay with low plasticity is estimated to be $c = 84\ kPa$ using an empirical correlation that relates c to SPT-N (Sowers).

The ultimate bearing capacity for a shallow foundation in the soil with the estimated shear strength parameters is calculated. Note that no surcharge term is included in the calculation of bearing capacity. Factored geotechnical bearing resistance at USL is calculated by applying the geotechnical resistance factor of $\Phi = 0.5$ for shallow foundation designs.

Design bearing pressure, q_{all}, at SLS for settlement not exceeding 25 mm is estimated based on equations by Terzaghi and Peck (1948, 1967) and recommendations provided by CFEM (2006) for preliminary design. The allowable bearing resistance at SLS condition is estimated for the very stiff clay layer with low plasticity. The modulus of vertical subgrade reaction for foundation design is estimated from an equation by Bowles, 1997. A summary of the results is provided in Table 13.

Table 13 – Recommended geotechnical resistance values and subgrade reaction modulus

BOREHOLE NO.	SPT – $N_{av.}$ (Blows/0.3 m)	DEVELOPMENT	q_{all} at SLS (kPa)	q_{ult} at ULS (kPa)	Φq_{ult} at ULS (kPa)	K_s (MN/m^3)
BH-1	38(0.76-3.5 mbgl)	North section of the subject site, within the footprint of the retail fuel pump island, canopy, and USTs	265	465	230	38.0
BH-2	40(2.29-3.51 mbgl)	Middle of the subject site, within the footprint of the commercial fuel pump island, canopy, and USTs	280	410	205	21.5
BH-3	18(0.76-1.98 mbgl)	South area of the subject site, within the footprint of the proposed future development	125	220	110	27.7
BH-4	26(1.52-1.98 mbgl)	Mid-west area of the subject site, within the footprint of the proposed Burger King restaurant building	180	315	160	32.8
BH-5	27(0.76-1.98 mbgl)	West area of the subject site, within the footprint of the proposed parking area	190	335	165	33.9
BH-6	26(1.52-1.98 mbgl)	Mid-west area of the subject site, within the footprint of the proposed C-Store and restaurant building	180	315	160	32.8
BH-7	32(0.76-1.22 mbgl)	Northwest corner area of the subject site, within the footprint of the proposed right-turn lane off of Hurontario Street	225	395	195	25.7
BH-8	17(0.76-2.74 mbgl)	South-middle area of the subject site, within the footprint of the proposed right-turn lane off of King Street.	120	210	105	24.6
*1.2m of soil cover must be maintained for all footings exposed to seasonal freezing conditions						

7.0 DESIGN DISCUSSION AND RECOMMENDATIONS

7.1 General Considerations

The comments provided in this report are intended only for the guidance of engineers, architects and contractors with a good knowledge of geotechnical designs. The numbers of boreholes investigated are within the recommended number for small sites that show consistent sub surface characteristics. Contractors and/or subcontractors bidding on or undertaking the work should, in this light be reasonably assured that conditions will not vary significantly. They may seek permission from owners to access the site for their own type of investigations, as well may make their own interpretations of the factual borehole results contained in this report. The following general comments are provided with respect to the conditions encountered and the intended scope of development.

7.2 Geotechnical Recommendations

The conditions recorded in the Boreholes indicate that footings founded at the prescribed elevation and lower would be constructed on stiff to very stiff clay. The recommended allowable bearing resistances for the building foundations at ULS and SLS are 522 kPa and 231 kPa, respectively, for conventional spread and/or strip footing foundations placed on these soils.

For any shallow structures; all exterior foundations and foundations in unheated areas must be provided with a minimum soil cover of 1.2 m or equivalent insulation for frost protection. The foundation depths recommended below are with respect to final grading levels, for Isolated Type of footings. It is recommended that all excavated footing bases must be evaluated by a qualified geotechnical engineer to ensure that the founding soils exposed at the excavation base are consistent with the design bearing pressure intended by the geotechnical engineer.

It should be noted that wet/dilatant soils may be encountered at the design foundation elevation at some borehole locations. Foundation subgrade comprising wet/dilatant soil will become weak and unstable due to disturbance, and will lose its integrity to support foundation. Consideration should be given to minimize disturbance to the foundation sub grade in these areas and the sub grade may need to be protected by a skim coat of lean concrete. For foundation excavations extending below the groundwater level, it will be necessary to lower and maintain the groundwater level below the excavation base. Further comments on the groundwater control are presented in Section 7.6 of this report.

7.3 Foundation

In general, any of the following options for types of footings are feasible; continuous, combined, spread, and raft or mat is a combined footing-slab that covers the entire area beneath a building and support all walls and columns. A raft foundation should be used (at least for economic reason) any time the individual footings would constitute half or more the area beneath a building.

Prior to pouring concrete for the footings, the footing sub grade should be cleaned of all deleterious materials such as topsoil, fill, softened, disturbed or caved materials, as well as any standing water. If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided. Native soils and engineered fill materials tend to weather rapidly and deteriorate on exposure to the atmosphere and surface water. Hence, foundation bases which remain open for an extended period of time should be protected by a skim coat of lean concrete.

The exposed sub grade should be proof-rolled to minimize differential settlement and to increase the bearing capacity. During the excavation, if loose material is found at the foundation level, the contractor is to remove all the loose material (until the dense soil is reached) and replace it with engineering fill granular material. Given this scenario, a conventional spread footing placed at this level should be founded on engineered fill if it is to have appropriate support. This engineered fill must consist of approved OPSS Granular B Type I (sand and gravel) materials compacted to 98% SPD. A grade raise may be considered. If this is the case, the proof rolled and compacted surface of the existing native soils will provide a satisfactory base for the placement and compaction of the engineered fill.

Provided that the foundation bases are not disturbed by excavation, surface water inflow, or freezing and thawing action, the following guidelines shall be followed in order for the engineered fill for the structure to be approved:

1. Fill containing construction debris (or similar) and organic matter inclusion should not be reused as backfill.
2. The excavated native soils and clean fill soils are considered suitable for backfill provided the water content of these soils are within 2 percent of the optimum moisture content. It should be noted that there may be zones within the silty sand soils that could be too wet to compact and, therefore, additional processing. Excavation and engineered fill may be required.
3. Continuous compaction testing will be performed on each layer of fill placed using nuclear gauge equipment. The on-site compaction test results will then be submitted to all relevant parties.
4. Once they have been placed and approved, the soils must be protected from excessive wetting, a minimum depth of 1.2m of soil cover or equivalent for frost protection is recommended for foundations founded on engineered fill.
5. A perimeter drain tile, leading to an outward discharge, should be placed at the exterior face of the foundation wall where any high-water table can cause freeze thaw damage or unacceptable infiltration to the foundation.

7.4 Slab-On-Grade Floor Using Engineered Fill

With a thickened edge slab on grade, the underside of the perimeter will be 400 mm to 500 mm below grade, and back fill will act as surcharge. To create a stable working surface and to distribute loadings, compacted OPSS 1010 Granular ‘A’ or equivalent should be placed below all floor slabs. The compacted OPSS 1010 Granular ‘A’ or equivalent should be 150 mm thick at minimum, compacted to 98 % of the standard density. Floor slabs below unheated buildings or equipment should be provided with adequate insulation to prevent cracking from potential frost heave unless the compacted Granular ‘A’ base is placed on clean limestone bedrock. A 100 mm thickness of high-density Styrofoam insulation, extending horizontally 1.8 m beyond the building/slab footprint, should be adequate to prevent frost heave where necessary.

For preliminary design, the test module of vertical sub grade reaction (K_s) for granular material over the encountered sub grade materials is presented below. These values should be modified by appropriate shape and depth factors to determine the vertical sub grade modulus (ks) for slabs and bases.

The top surface of the sub grade below slabs will be composed of approved OPSS Granular B engineered fill, compacted to 98% Standard Proctor maximum dry density. A minimum of 150 mm of granular A to 100% SPD will be placed over the granular B.

Table 14 – Modulus of Vertical Subgrade Reaction

Subgrade Material	Modulus of Vertical Subgrade Reaction, K_s (MN/m3)
Engineered Fill 23	25.0

7.5 Earthquake Design Parameters

The 2012 Ontario Building Code (OBC) stipulates the methodology for earthquake design analysis, as set out in Subsection 4.18.7. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration and the site classification.

The parameters for determination of the Site Classification for Seismic Site Response are set out in Table 4.1.8.4.A of the 2012 OBC. The classification is based on the determination of the average shear wave velocity in the top 30 metres of the site stratigraphy, where shear wave velocity (V_s) measurements have been taken. In the absence of such measurements, the classification is estimated on the basis of empirical analysis of un-drained shear strength or penetration resistance. The applicable penetration resistance is that which has been corrected to a rod energy efficiency of 60% of the theoretical maximum or the (N_{60}) value.

Based on the borehole information, the subsurface stratigraphy generally comprises of stiff soil ($15 \leq N_{60} \leq 50$). On this basis, the site designation for seismic analysis is ***Class D*** according to Table 4.1.8.4.A from the quoted code.

The site specific 5% damped spectral acceleration coefficients, and the peak ground acceleration factors are provided in the 2012 OBC - Supplementary Standard SB-1 (September 14, 2012), Table 1.2, location Caledon, Ontario.

7.6 Groundwater Control

Based on the groundwater measurements, excavations for all foundations and utilities may be extended below the groundwater table. As described in the project proposed development, the water level - depend on the seasonal conditions. Therefore, the excavation area and foundation zone maybe be in a wet area. Dewatering by using well points should be considered to make the construction area dry. As the groundwater at the site may fluctuate seasonally it can be expected to be even higher in response to major precipitation events, no impact to the development is expected.

Backfilling of foundations shall be carried out with approved OPSS Granular B material provided it can be placed in maximum 300mm (1 ft.) loose lifts and compacted to a minimum of 98% Standard Proctor maximum dry density. It is important that backfilling of wall and grade beam foundations must take place on both sides at the same time to avoid any unbalanced lateral loading. As previously specified, backfilling of under slab interior excavations must be made with approved OPSS Granular B Type I (sand and gravel) material, placed in 300 mm loose lifts and compacted to at least 98% Standard Proctor maximum dry density and 150 mm of granular A to 100% SPD. Backfilling of service trenches under proposed pavement areas shall be carried out using approved imported soils or imported OPSS approved Granular B materials provided it can be placed in maximum 300mm (12 inch) loose lifts and compacted to a minimum of 98% Standard Proctor maximum dry density.

The onsite fill materials may not meet compaction requirements or may contain substantial amounts of silt or clay and therefore, are not considered suitable to be used as backfill. Therefore, it is expected that most material will have to be imported. Prior to importing any material from other sites to this site for use as backfill for the parking lot, such as utility trenches, the material must satisfy the following two conditions:

(a) Proctor compaction tests must show that the soil is capable of being compacted to a satisfactory density; results submitted to A&A for approval and then be delivered on site within 2% of its optimum moisture content.

(b) Materials that have been imported and approved for use that are stored onsite should be maintained within 2% of their optimum moisture content. They should also be protected from the weather with tarps.

7.7 Canopy

The proposed gas station canopy may be founded on the natural between 1.2 m and 3 m depth below the existing ground surface and designed for (enhanced) SLS bearing pressure and factored geotechnical resistance at ULS of 350 kPa and 600 kPa respectively. Since the canopy will not be heated, and snow will be removed from its vicinity, a minimum of 2.4 m of earth cover or equivalent insulation should be provided to the footings to protect them from damage due to frost penetration.

7.8 Convenience store

The convenience store may also can be founded on the natural, between 1.2 m and 3 m depth below the existing ground surface and designed for SLS bearing pressure and existing geotechnical resistance at ULS of 550 kPa and 950 kPa respectively. A minimum of 1.5 m of earth cover or equivalent insulation should be provided to the footing to protect them from damage due to frost penetration.

7.9 Installation of Underground Tanks

It is understood that two underground fuel storage tanks of 65m^3 capacity with 3m ϕ for each will be installed at a depth of 4.5m to 5m approximately below the existing ground surface. These tanks may be set on a concrete slab or on 300 mm thick Granular A engineered fill pad compacted to 100 percent standard Proctor maximum dry density.

The proposed serviceability limit state (SLS) bearing pressure at the proposed founding level of the tanks shall be 530 kPa. The existing geotechnical resistance at ULS is 925 kPa. The settlements of the tanks are expected to be within the normally tolerated limits of 25 mm total and 19 mm differential movements. However, the established groundwater table is anticipated at a depth of 1.5m below the existing ground surface. The groundwater table is subject to seasonal fluctuations and may be at a higher level during wet weather periods. The conventional practice is to design the tanks when empty to be capable of withstanding the hydrostatic uplift pressures corresponding to the groundwater table being at the ground surface. The magnitude of the hydrostatic uplift may be calculated using the following formula:

$$P = \gamma_w d$$

Where:

P = hydrostatic uplift pressure acting on the base of the structure (kPa)

$\boldsymbol{\gamma_w}$ = unit weight of water (9.8 kN/m3)

d = depth of base of structure below the design high water level (m)

The resistance of gross uplift of the structure can be increased by simply increasing the mass of the structure.

7.10 Site Grading and Engineered Fill Construction

Site grading operations involving "cut and fill" procedures in the order of 6± mare expected throughout the site. It is recommended to construct engineered fill in areas to be raised in order to suitably support the future roadway, infrastructure servicing and lightly loaded building structures.

The surficial topsoil layer varied in thickness between 150 and 425 mm at the borehole locations. It should be noted that the thickness of the organic soil layer could vary drastically across the site from those reported at the borehole locations.

It is noted that topsoil stripping operations should be conducted when the ground is not wet and will support large scale construction equipment. Over-stripping can result when the ground conditions are wet and unstable.

Inorganic onsite native soil deposits from potential "cut" areas may potentially be reused to construct engineered fill capable of supporting building structures, infrastructure servicing and future roadways. The natural moisture content of the "cut" soils to be used as engineered fill should be within 3% below their optimum moisture contents to achieve the specified degree of compaction.

Any shortfall of fill material required for site grading operations may be made with similarly graded imported soils for the various purposes described above. It is recommended that any proposed borrow source materials be tested prior to importing, in order to ensure that the environmental quality of the imported fill meets all environmental approval criteria and to ensure that the natural moisture content of the fill is suitable for compaction.

It is recommended that engineered fill construction be conducted during the summer and early fall months when drier warmer weather conditions typically exist as the onsite soils are sensitive to moisture and will become difficult to handle and compact to the specified degree of compaction when wet.

The onsite deposits are frost-susceptible. Constructing engineered fill, backfilling footings, foundation walls and service trenches using these finer grained soils during the winter months is not advisable, unless suitable weather conditions prevail, the soils are at suitable moisture content, and strict procedures are followed and monitored on a full-time basis by the geotechnical engineer.

The onsite soils are susceptible to softening and deformation when exposed to excessive moisture and construction traffic. As a result, it is imperative that the grading/filling operations are planned and maintained to direct surface water run-off to low points and then be positively drained by suitable means. During periods of wet weather, construction traffic should be directed along the designated construction routes so as not to disturb and rut the exposed subgrade soil. Temporary construction roads consisting of clear crushed material (such as crushed stone or recycled concrete) may be required during poor weather conditions such as wet spring or fall.

7.11 Underground Site Servicing

It is anticipated that municipal water-main and sewer servicing will generally be in the range of 2 to 4 m below final design grades.

7.11.1 Excavation Conditions

Trenching can be carried out using conventional open cut procedures. The excavations will generally intersect native and/or re-compacted fill soils. The native and re-compacted fill soil will generally provide suitable subgrade support to sewer and watermain serving. Any loose, unstable and/or organic soils encountered at the pipe invert should be sub-excavated and replaced with well compacted Granular "A" which should be placed in 150 mm thick layers and compacted to at least 95% Standard Proctor Maximum Dry Density (SPMDD). The support of pipes in these areas can also be achieved with non-shrinkable fill, if poor soil is encountered at the subgrade level and fully removed.

Excavation side slopes should comply with the current "Regulations for Construction Projects under the Ontario Occupational Health and Safety Act". The native or re-compacted fill soils can be generally classified as Type 3 soils. Excavation in the Type 3 soils should be cut to side slopes of 1H: 1V throughout. The excavation side slopes should be suitably protected from erosion processes. Should unstable and/or wet conditions be encountered, side slopes are to be flattened to a stable configuration. The geotechnical engineer should be retained to examine and inspect cut slopes to ensure construction safety.

7.11.2 Pipe Bedding

Any unsuitable soils exposed at the pipe subgrade should be sub-excavated and replaced with imported Granular "A", placed in thin layers and compacted to at least 95% SPMDD, or can be removed and supported on non-shrinkable fill. The bedding requirements for the services should be in accordance with Ontario Provincial Standard Drawings OPSD - 802 for flexible and rigid pipes. The bedding shall be a Class "B" and consist of at least 150 mm thick Granular "A" compacted to at least 95% SPMDD. Granular "A" should be used to backfill around the pipe to at least 150 mm above the top of the pipe. Particular attention should be given to ensure material placed beneath the haunches of the pipe is adequately compacted. Recycled asphalt will not be allowed to be used in Granular "A" bedding material.

7.11.3 Trench Backfill

Excavated inorganic materials are considered suitable for reuse as trench backfill. If necessary, potential mixing of drier and wetter excavated soils in proper ratios can be done to produce a suitable mixture near the material's optimum moisture content in order to achieve the required compaction specification. Conversely, judicious addition of water may be required if the soils are significantly drier than their optimum moisture content in order to facilitate suitable compaction.

The backfill should be placed in thin layers, 300 mm thick (or less dependents on the demonstrated success of compaction based on in-situ density test results) and compacted to no less than 95% SPMDD. Other types of materials such as organic soils, overly wet soils, boulders and frozen materials (if work is carried out in the winter months) should not be used for backfilling. Backfilling operations should follow closely after excavation so that only a minimal length of trench slope is exposed at any one time so as to minimize potential problems. This will potentially minimize over-wetting of the subgrade material. Particular attention should be given to make sure frozen material is not used as backfill should construction extend into the winter season.

7.11.4 Curbs and Sidewalks

The concrete for any new exterior curbs and sidewalks should be proportioned, mixed placed and cured in accordance with the requirements of OPSS 353, OPSS 1350 and the Municipality. During cold weather, the freshly placed concrete should be covered with insulating blankets to protect against freezing. The subgrade for the sidewalks should consist of undisturbed natural soil or well compacted fill. A minimum 100mm thick layer of compacted (minimum 98 percent SPMDD) Granular 'A' is recommended below sidewalk slabs.

7.12 Pavement Structures (Parking Areas and Access Roads)

The pavement areas shall be constructed on an adequately prepared sub grade consisting of firm to stiff, sandy silty clay fill and/or Granular B. The sub grade must be stripped of all asphalt, loose, wet and deleterious materials and then proof rolled and compacted under geotechnical supervision to a minimum of 1700kN/m^3 as tested by nuclear densimeter. Provided that the preparation of the site is completed in accordance with the recommendations stated above, the design detailed below will be suitable for proposed access roads and parking lots. The following minimum granular base recommendations should provide acceptable performance.

Table 15 – Minimum Granular Base Recommendations as per OPSS

Paved Area Type	Compaction Requirements	Material Requirements
Travel Lanes	Compacted to 98% SPMDD	150 mm of OPSS Granular A
Parking Lots	Compacted to 98% SPMDD	150 mm of OPSS Granular A
Access Roads	Compacted to 98% SPMDD	250 mm of OPSS Granular A

Subgrade conditions are critical regarding the long-term performance of the surface and the subgrade should be prepared in accordance with Section 5.1.1. The thickness of the granular base could be increased at the discretion of the design Engineer, or granular sub-base layers added, to accommodate site conditions at the time of construction.

The finished surfaces of access roads and parking areas should be sloped at 2 % or greater to promote runoff to designated surface drainage and catch basins. Any weak or soft areas encountered at the original surface must be further sub-excavated and replaced by engineered fill. A biaxial geogrid type BX2000 by Terrafix (or equivalent) shall be placed directly above the surface of the existing exposed sub grade. The sub grade must then be raised to the desired level using approved imported materials as specified. Stringent compaction and placement control procedures shall be maintained to ensure uniform sub grade moisture and density conditions are achieved. Based on the sub grade characteristics and normal anticipated traffic loading, the pavement structures noted in Table 16 are recommended:

Table 16 – Recommended Pavement Thicknesses for Light and Heavy Duty

Recommended Pavement Thicknesses		
Material	**Light Duty**	**Heavy Duty**
HL3 Surface Asphalt	40mm	40mm
HL8 Binder Asphalt	50mm	80mm
Granular Base course OPSS Granular "A"	150mm	150mm

8.0 LIMITATIONS OF REPORT

This report has been prepared for Bikram Dhillon representing BVD Petroleum (the Client), who retained the services of A&A to conduct a geotechnical investigation for a proposed development on a property located at 14027 Hurontario Street, Caledon, Ontario. Further dissemination of this report is not permitted without A&A’s prior written approval. A&A has carefully assessed all information provided to them during this investigation but makes no guarantees or warranties as to the accuracy or completeness of this provided information.

The comments given in this report are intended only for the guidance of design engineers and Architects. Contractors bidding on or undertaking the work, should in this light, decide that further field investigations, and interpretations of the factual borehole results are necessary to draw their own conclusions as to how the subsurface conditions may affect them. Should soil conditions during excavation for the foundations prove to be different than what have been described in this report, the author of this report should be notified as soon as possible. No liability or claims may be made by owners or third parties against A&A for factors outside (A&A's) control. An independent quality control firm, such as Terraprobe, must be made available for all concrete and compaction testing associated with construction. All testing results should be made available to the owner, designers, consultant and general contractor.

The site investigation and recommendations follow generally accepted practice for Geotechnical Consultants in Ontario. Materials testing has been completed in accordance with ASTM or CSA Standards or modifications of these standards that have become standard practice.

SIGNED:

November 1, 2019
Saad Al-Dabbagh, P. Eng

9.0 REFERENCES

Bowles, & E., J. (1996). *Foundation Analysis and Design.* McGraw Hill Inc.

Canadian foundation engineering manual. 4th Edition. (2006). Richmond, B.C : Canadian Geotechnical Society.

Sowers, G. (1979). *Introductory Soil Mechanics and Foundations: Geotechnical Engineering.* New York: MacMillan.

Terzaghi, K., & Peck, R. (1967). *Soil Mechanics in Engineering Practice.* New York: John Wiley.

APPENDIX A – Site Drawings

Figure 1 – Site Location Map for The Subject Site at 14027 Hurontario Street, Caledon, ON

A & A ENVIRONMENTAL CONSULTANTS INC.
16 Young St., Woodstock, ON, N4S 3L4
Tel: 519 266-4680

Location Map Indicating The Subject Site at 14027 Hurontario Street, Inglewood, ON

400 m

Project #: 4545-BVD Caledon
September 2019

N E S W

Figure 2 – Topographic Map of Subject Study Area

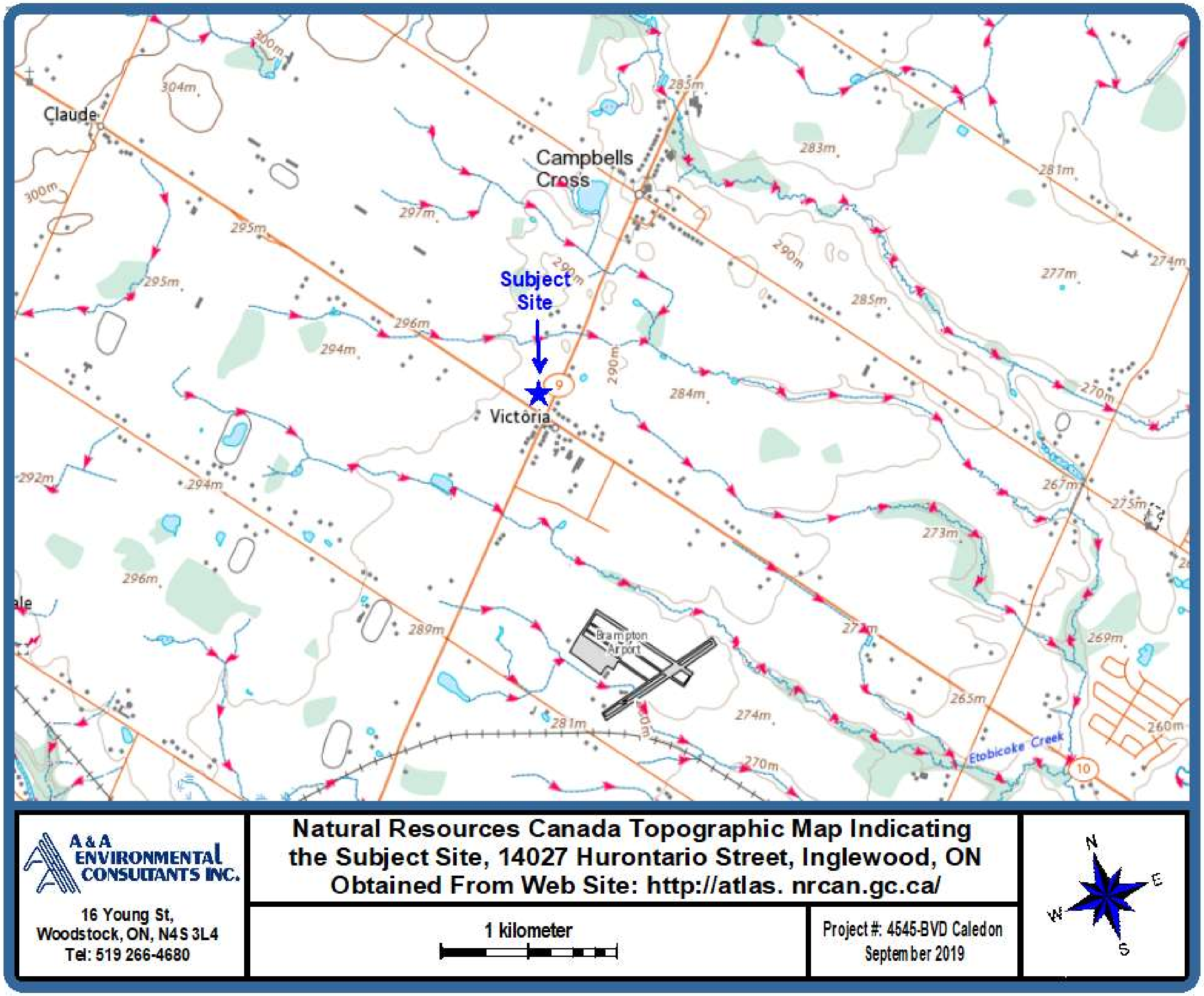


Figure 3 – Geotechnical Boreholes Location, Site Image

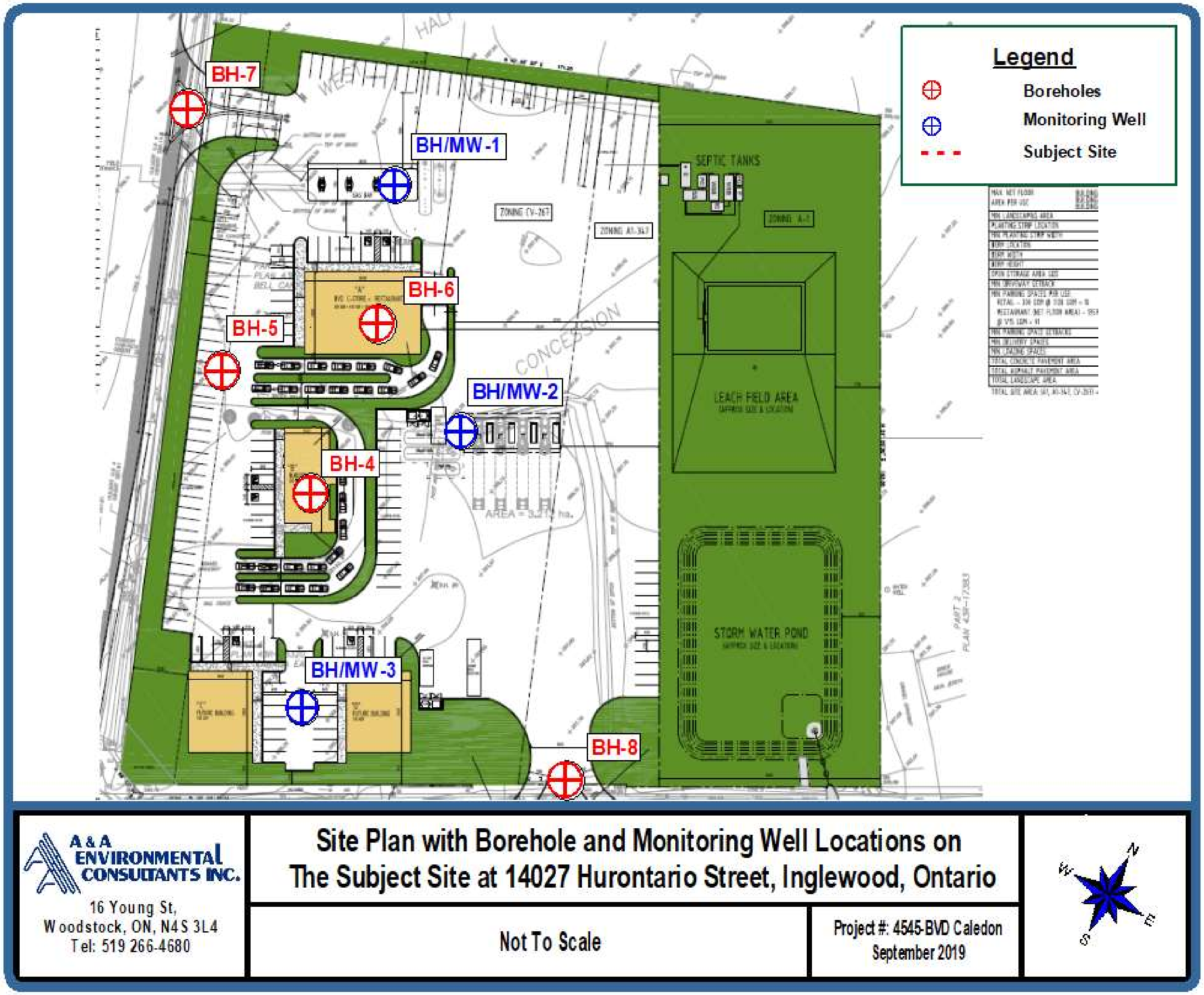
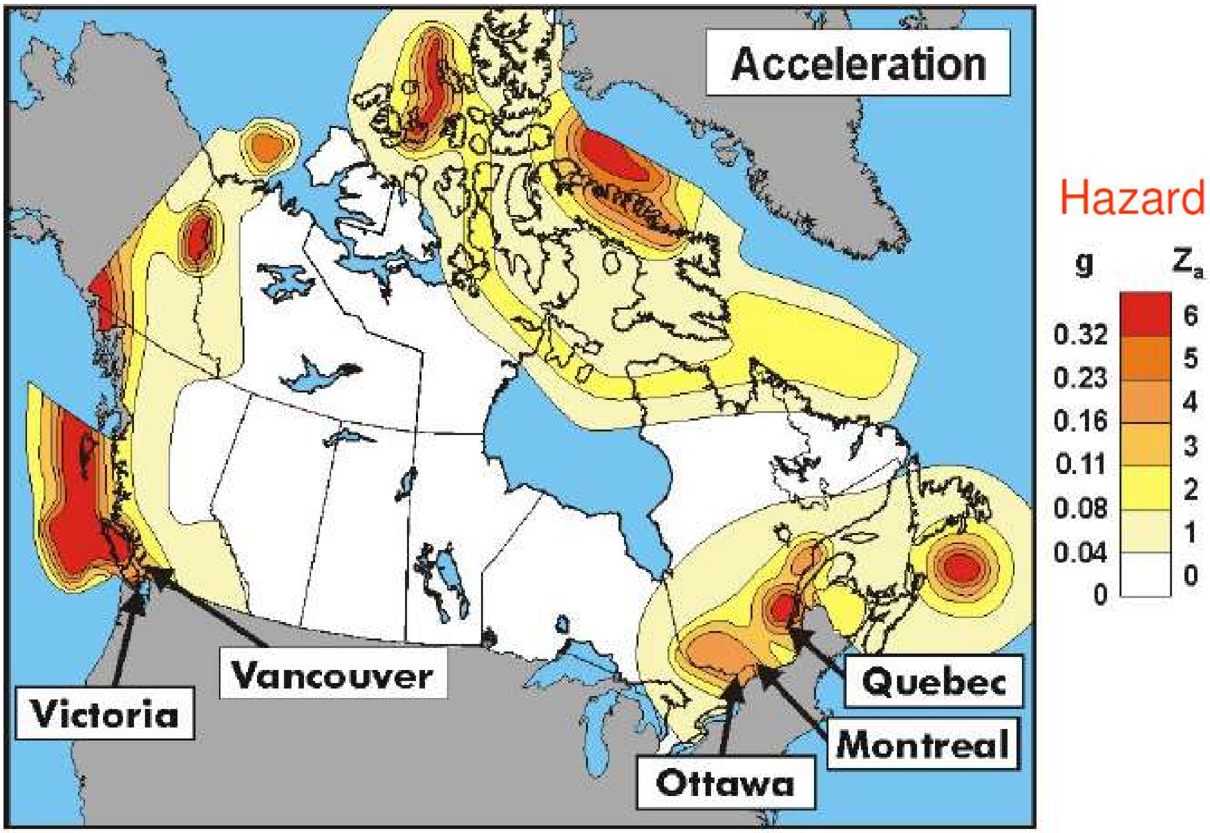


Figure 4 – Geotechnical Boreholes Location, Satellite Image

Figure 5 – Earthquake Zoning Hazards



APPENDIX B – Borehole Logs and Explanation of Terms and Symbols

Explanation of Terms and Symbols

The terms and symbols used on the borehole logs to summarize the results of field investigation and subsequent laboratory testing are described in these pages.

Test Data

Abbreviations, graphic symbols and relevant test method designations are as follows:

W_L Liquid Limit

W_P Plastic Limit

I_P Plasticity Index

Soils are classified and described according to their engineering properties and behaviours.

RELATIVE DENSITY OF GRANULAR SOILS

SPT N Value (Blows/300mm)	Relative Density	Angle of Internal Friction *
0 - 4	Very loose	< 30 °
4 - 10	Loose	30 ° - 35 °
10 - 30	Medium dense	35 ° - 40 °
30 - 50	Dense	40 ° - 45 °
> 50	Very dense	> 45 °

* After Meyerhof

ROCK STRENGTH CLASSIFICATION

Unconfined Compressive Strength (MN/m^2)	Description
< 1.25	Very weak
1.25 - 5	Weak
5 - 12.5	Moderately Weak
12.5 - 50	Moderately Strong
50 - 100	Strong
100 - 200	Very Strong
> 200	Extremely Strong

CONSISTENCY OF COHESIVE SOILS

Consistency	Undrained Shear Strength (kPa)	Spt N-Index (blows/0.3m)
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

PROJECT: Geotechnical & Hydrogeological	**BH LOCATION:** Northeast portion of subject site	**BOREHOLE NO: BH/MW1**
PROJECT NO: 4545- BVD Caledon	**LOCATION:** NE corner of Hurontario Street & King Street Intersection, Caledon, ON	
PROJECT MANAGER: T. Demers	**COMPANY NAME:** A&A Environmental Consultants Inc.	

SAMPLE TYPE	SHELBY TUBE	CORE SAMPLE	SPT SAMPLE	GRAB SAMPLE	NO RECOVERY
BACKFILL TYPE	BENTONITE	PEA GRAVEL	SLOUGH	GROUT	DRILL CUTTINGS

DEPTH (ft)	DEPTH (m)	SOIL PROFILE	Soil Description	SAMPLE TYPE	SAMPLE NO	BLOWS / 150 mm
			Ground Surface			
0.0	0.0		Topsoil			
1.0			Silty Sand Light brown, dry, no odour.		1	
2.0					2	
3.0	1.0		Clayey Silt Brown with grey mottling, damp, no odour.		3	
4.0					4	
5.0						
6.0	2.0				5	
7.0					6	
8.0			Wet.		7	
9.0						
10.0	3.0				8	
11.0					9	
12.0						
13.0	4.0					
14.0						
15.0						
16.0	5.0				10	
17.0						
18.0						
19.0						
20.0	6.0		End of Log			
21.0						
22.0						
23.0	7.0					
24.0						
25.0						

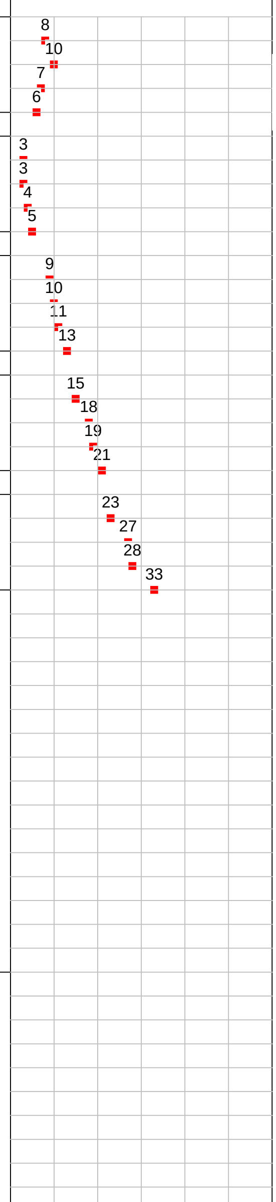
Notes:
- Borehole augered to 20 feet to facilitate monitoring well installation.

Blowcount

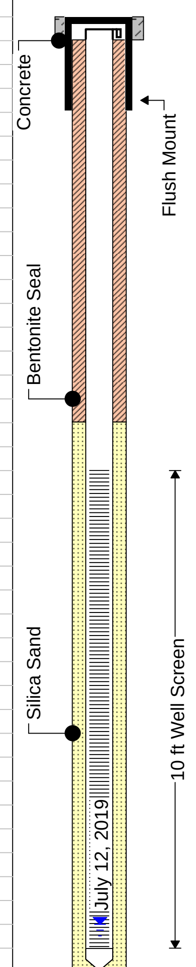
10 20 30 40 50

PPM

100 200 300 400



Monitoring Well



Notes

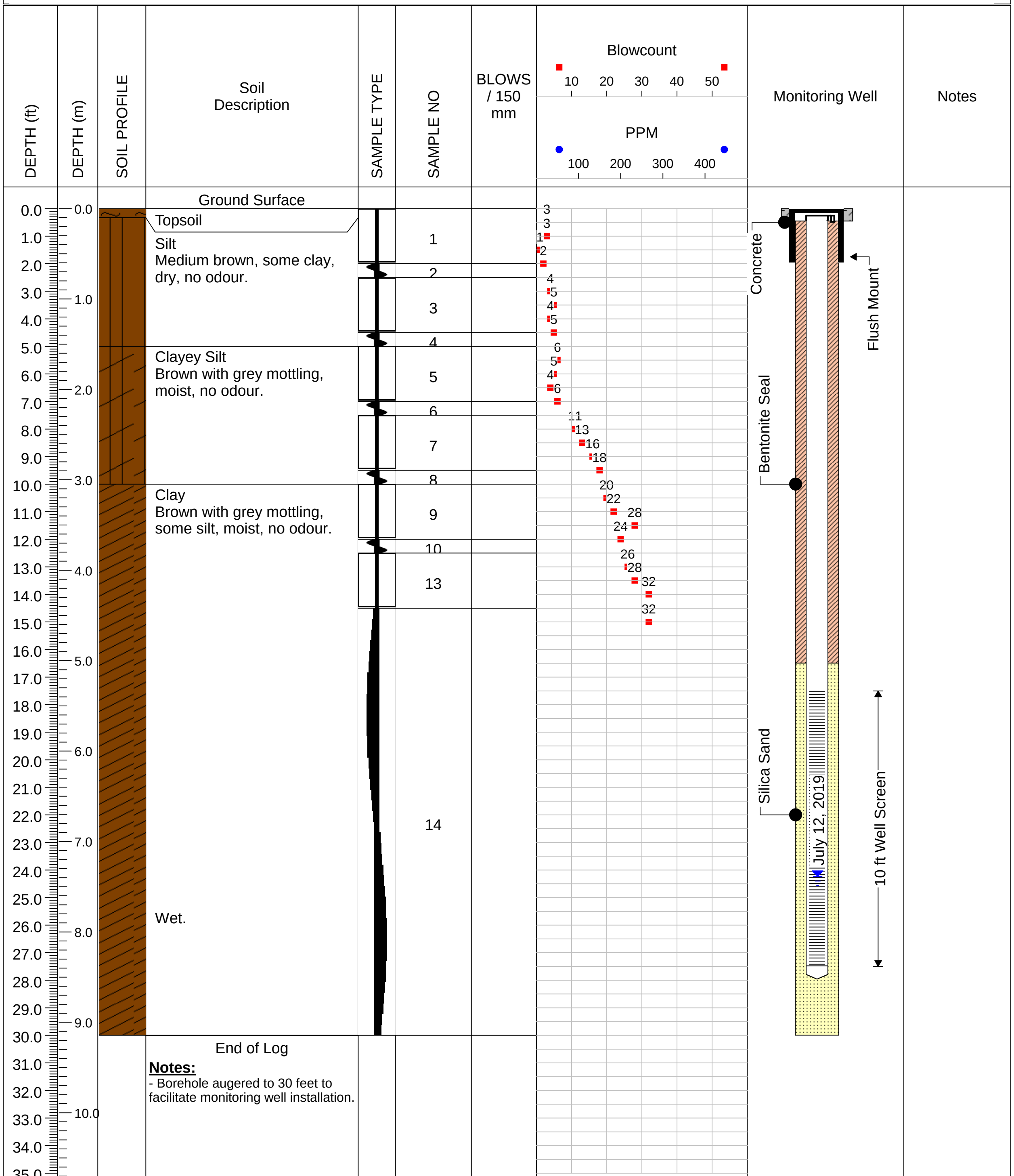
A & A Environmental Consultant Inc.

16 Young Street Woodstock, ON

LOGGED BY: T. Thornton	**COMPLETION DEPTH:** 20 feet
REVIEWED BY: A. Rasoul	**DRILL METHOD:** Split spoon & rotary
DRILL DATE: July 12, 2019	**Page:** 1 of 1

PROJECT: Geotechnical & Hydrogeological	**BH LOCATION:** Central south portion of subject site	**BOREHOLE NO: BH/MW2**
PROJECT NO: 4545- BVD Caledon	**LOCATION:** NE corner of Hurontario Street & King Street Intersection, Caledon, ON	
PROJECT MANAGER: T. Demers	**COMPANY NAME:** A&A Environmental Consultants Inc.	

SAMPLE TYPE	SHELBY TUBE	CORE SAMPLE	SPT SAMPLE	GRAB SAMPLE	NO RECOVERY
BACKFILL TYPE	BENTONITE	PEA GRAVEL	SLOUGH	GROUT	DRILL CUTTINGS



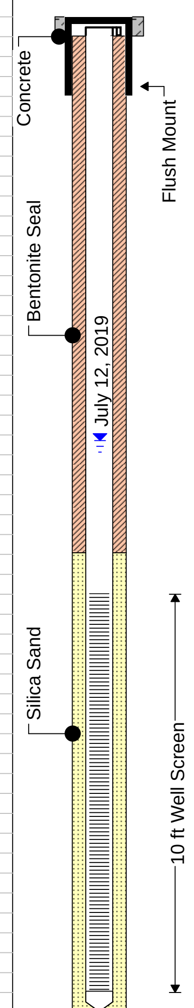
A & A Environmental Consultant Inc. 16 Young Street Woodstock, ON	**LOGGED BY:** T. Thornton	**COMPLETION DEPTH:** 30 feet
	REVIEWED BY: A. Rasoul	**DRILL METHOD:** Split spoon & rotary
	DRILL DATE: July 12, 2019	**Page:** 1 of 1

PROJECT: Geotechnical & Hydrogeological	**BH LOCATION:** South portion of subject site
BOREHOLE NO: BH/MW3	
PROJECT NO: 4545- BVD Caledon	**LOCATION:** NE corner of Hurontario Street & King Street Intersection, Caledon, ON
PROJECT MANAGER: T. Demers	**COMPANY NAME:** A&A Environmental Consultants Inc.

SAMPLE TYPE	SHELBY TUBE	CORE SAMPLE	SPT SAMPLE	GRAB SAMPLE	NO RECOVERY
BACKFILL TYPE	BENTONITE	PEA GRAVEL	SLOUGH	GROUT	DRILL CUTTINGS

DEPTH (ft)	DEPTH (m)	Soil Description	SAMPLE NO	BLOWS / 150 mm	Blowcount (10 20 30 40 50) / PPM (100 200 300 400)	Monitoring Well	Notes
0.0	0.0	Ground Surface					
0.0 – 0.5		Topsoil					
0.5 – 2.5		Gravelly Silty Sand Medium brown, dry, no odour.	1		5, 10, 12, 5		
			2				
2.5 – 7.5	1.0	Silt Medium brown, damp, no odour.	3		3, 3, 4, 9		
			4				
	2.0	Brown with grey mottling, some clay, dry.	5		12, 12, 16, 19		
			6				
7.5 – 15.0		Clayey Silt Brown with grey mottling, trace gravel, dry, no odour.	7		22, 21, 23, 26		
	3.0		8				
			9		25, 25, 25, 29		
			10				
	4.0	Damp.	11		24, 30, 33, 34		
15.0 – 25.0	5.0 – 7.0	Clay Brown with grey mottling, some silt, moist, no odour.	12				
25.0		End of Log					

Notes:
- Borehole augered to 25 feet to facilitate monitoring well installation.



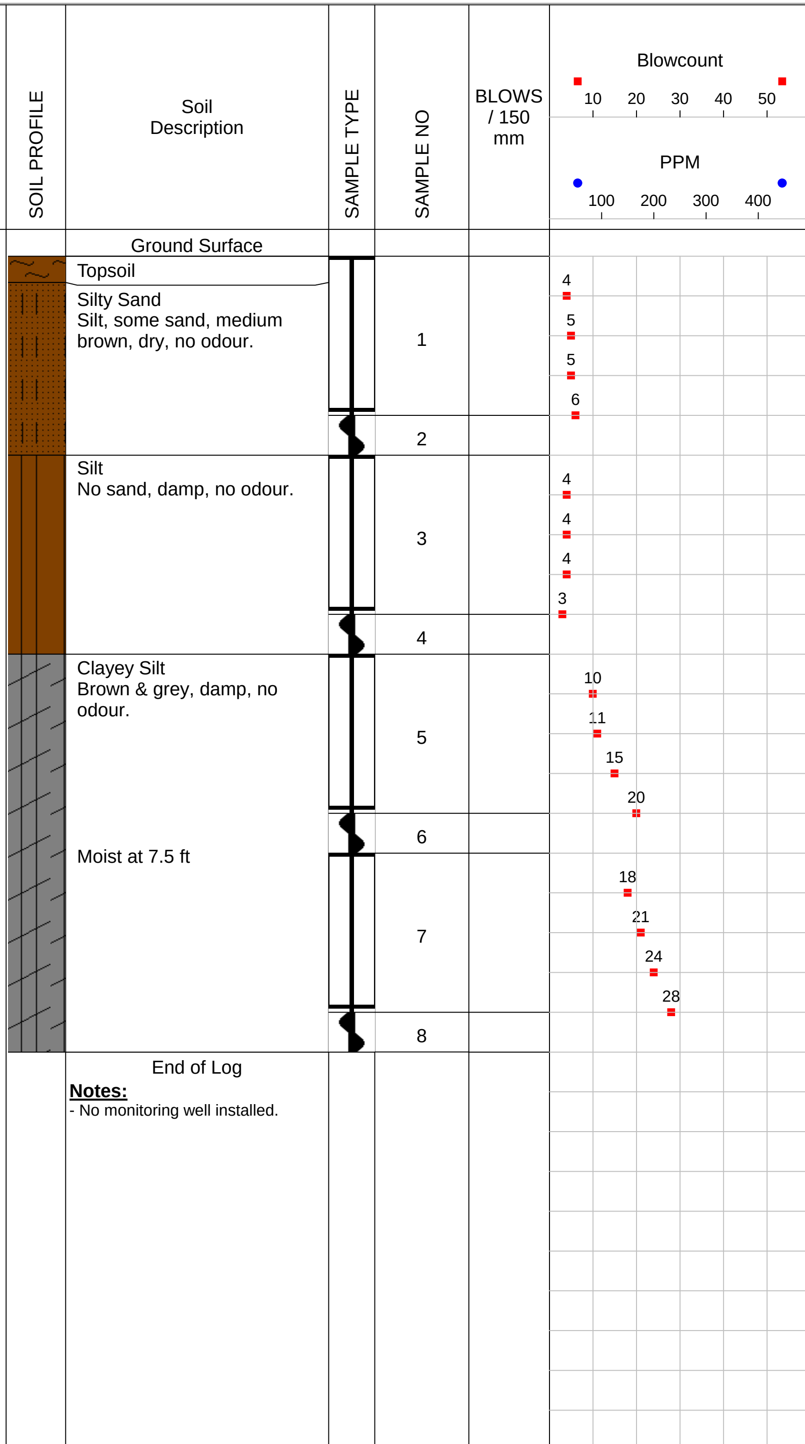
A & A Environmental Consultant Inc.
16 Young Street Woodstock, ON

LOGGED BY: T. Thornton	**COMPLETION DEPTH:** 25 feet
REVIEWED BY: A. Rasoul	**DRILL METHOD:** Split spoon & rotary
DRILL DATE: July 12, 2019	**Page:** 1 of 1

PROJECT: Geotechnical & Hydrogeological	**BH LOCATION:** Central portion of subject site	**BOREHOLE NO: BH4**
PROJECT NO: 4545- BVD Caledon	**LOCATION:** NE corner of Hurontario Street & King Street Intersection, Caledon, ON	
PROJECT MANAGER: T. Demers	**COMPANY NAME:** A&A Environmental Consultants Inc.	

SAMPLE TYPE	SHELBY TUBE	CORE SAMPLE	SPT SAMPLE	GRAB SAMPLE	NO RECOVERY
BACKFILL TYPE	BENTONITE	PEA GRAVEL	SLOUGH	GROUT	DRILL CUTTINGS

DEPTH (ft)	DEPTH (m)	SOIL PROFILE	Soil Description	SAMPLE TYPE	SAMPLE NO	BLOWS / 150 mm	Monitoring Well	Notes
			Ground Surface					
0.0	0.0		Topsoil					
			Silty Sand Silt, some sand, medium brown, dry, no odour.		1			
2.0					2			
			Silt No sand, damp, no odour.		3			
3.0	1.0							
					4			
5.0			Clayey Silt Brown & grey, damp, no odour.		5			
	2.0				6			
			Moist at 7.5 ft		7			
					8			
10.0	3.0		End of Log					
			Notes: - No monitoring well installed.					
13.0	4.0							
15.0								



A & A Environmental Consultant Inc. 16 Young Street Woodstock, ON	**LOGGED BY:** T. Thornton	**COMPLETION DEPTH:** 10 feet
	REVIEWED BY: A. Rasoul	**DRILL METHOD:** Split spoon
	DRILL DATE: July 12, 2019	**Page:** 1 of 1

PROJECT: Geotechnical & Hydrogeological	**BH LOCATION:** Central W boundary	**BOREHOLE NO: BH5**
PROJECT NO: 4545- BVD Caledon	**LOCATION:** NE corner of Hurontario Street & King Street Intersection, Caledon, ON	
PROJECT MANAGER: T. Demers	**COMPANY NAME:** A&A Environmental Consultants Inc.	

SAMPLE TYPE	SHELBY TUBE	CORE SAMPLE	SPT SAMPLE	GRAB SAMPLE	NO RECOVERY
BACKFILL TYPE	BENTONITE	PEA GRAVEL	SLOUGH	GROUT	DRILL CUTTINGS

DEPTH (ft)	DEPTH (m)	SOIL PROFILE	Soil Description	SAMPLE TYPE	SAMPLE NO	BLOWS / 150 mm	Blowcount (10 20 30 40 50) / PPM (100 200 300 400)	Monitoring Well	Notes
0.0	0.0		Ground Surface						
0.0			Gravel						
0.5			Silt Medium brown, dry, no odour, trace gravel.		1		9		
1.0							6		
1.5							5		
2.0							6		
2.0–2.5					2				
3.0	1.0				3		9		
3.5							10		
4.0							12		
4.5							14		
4.5–5.0					4				
5.0			Clayey Silt Silt, some clay, brown & grey, dry, no odour, trace gravel.		5				
5.5							14		
6.0							14		
6.5	2.0						17		
7.0							19		
7.0–7.5					6				
8.0					7		17		
8.5							17		
9.0							21		
9.5							22		
9.5–10.0	3.0				8				
10.0			End of Log						

Notes:
- No monitoring well installed.

Depth scale (ft): 0.0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0 11.0 12.0 13.0 14.0 15.0; (m): 0.0 1.0 2.0 3.0 4.0

A & A Environmental Consultant Inc.
16 Young Street Woodstock, ON

LOGGED BY: T. Thornton	**COMPLETION DEPTH:** 10 feet
REVIEWED BY: A. Rasoul	**DRILL METHOD:** Split spoon
DRILL DATE: July 12, 2019	**Page:** 1 of 1

PROJECT: Geotechnical & Hydrogeological	**BH LOCATION:** NW portion of site	**BOREHOLE NO: BH6**
PROJECT NO: 4545- BVD Caledon	**LOCATION:** NE corner of Hurontario Street & King Street Intersection, Caledon, ON	
PROJECT MANAGER: T. Demers	**COMPANY NAME:** A&A Environmental Consultants Inc.	

SAMPLE TYPE	SHELBY TUBE	CORE SAMPLE	SPT SAMPLE	GRAB SAMPLE	NO RECOVERY
BACKFILL TYPE	BENTONITE	PEA GRAVEL	SLOUGH	GROUT	DRILL CUTTINGS

DEPTH (ft)	DEPTH (m)	SOIL PROFILE	Soil Description	SAMPLE TYPE	SAMPLE NO	BLOWS / 150 mm	Blowcount (10 20 30 40 50) / PPM (100 200 300 400)	Monitoring Well	Notes
			Ground Surface						
0.0	0.0		Topsoil						
0.5			Clayey Silt Silt, some clay, brown & grey, dry, no odour.		1		3		
1.0							4		
1.5							4		
2.0							4		
2.5					2				
3.0	1.0				3		4		
3.5							5		
4.0							5		
4.5			Moist at 4.5 ft				7		
5.0					4				
5.5					5		11		
6.0							12		
6.5	2.0						14		
7.0							17		
7.5					6				
8.0					7		12		
8.5							14		
9.0							20		
9.5	3.0				8		21		
10.0			End of Log **Notes:** - No monitoring well installed.						
11.0									
12.0									
13.0	4.0								
14.0									
15.0									

A & A Environmental Consultant Inc.
16 Young Street Woodstock, ON

LOGGED BY: T. Thornton	**COMPLETION DEPTH:** 10 feet
REVIEWED BY: A. Rasoul	**DRILL METHOD:** Split spoon
DRILL DATE: July 12, 2019	**Page:** 1 of 1

PROJECT: Geotechnical & Hydrogeological	**BH LOCATION:** Along Hurontario St Rd	**BOREHOLE NO: BH7**
PROJECT NO: 4545- BVD Caledon	**LOCATION:** NE corner of Hurontario Street & King Street Intersection, Caledon, ON	
PROJECT MANAGER: T. Demers	**COMPANY NAME:** A&A Environmental Consultants Inc.	

SAMPLE TYPE	SHELBY TUBE	CORE SAMPLE	SPT SAMPLE	GRAB SAMPLE	NO RECOVERY
BACKFILL TYPE	BENTONITE	PEA GRAVEL	SLOUGH	GROUT	DRILL CUTTINGS

DEPTH (ft)	DEPTH (m)	SOIL PROFILE	Soil Description	SAMPLE TYPE	SAMPLE NO	BLOWS / 150 mm	Blowcount (10 20 30 40 50) / PPM (100 200 300 400)	Monitoring Well	Notes
			Ground Surface						
0.0	0.0		Gravel						
			Gravelly Sand Medium brown, dry, no odour.						
1.0					1		21, 32, 29, 29		
2.0					2				
3.0	1.0				3		24, 16, 16, 20		
4.0					4				
5.0			Silty Clay Dark brown & grey, moist, no odour.						
6.0	2.0				5		26, 21, 10, 2		
7.0					6				
8.0					7		2, 7, 3, 6		
9.0	3.0				8				
10.0			End of Log						
11.0									
12.0									
13.0	4.0								
14.0									
15.0									

Notes:
- No monitoring well installed.

PROJECT: Geotechnical & Hydrogeological	**BH LOCATION:** Along King St Rd	**BOREHOLE NO: BH8**
PROJECT NO: 4545- BVD Caledon	**LOCATION:** NE corner of Hurontario Street & King Street Intersection, Caledon, ON	
PROJECT MANAGER: T. Demers	**COMPANY NAME:** A&A Environmental Consultants Inc.	

SAMPLE TYPE	SHELBY TUBE	CORE SAMPLE	SPT SAMPLE	GRAB SAMPLE	NO RECOVERY
BACKFILL TYPE	BENTONITE	PEA GRAVEL	SLOUGH	GROUT	DRILL CUTTINGS

DEPTH (ft)	DEPTH (m)	Soil Description	SAMPLE NO	BLOWS / 150 mm	Blowcount (10 20 30 40 50) / PPM (100 200 300 400)	Monitoring Well	Notes
0.0	0.0	Ground Surface Gravel					
0.5		Gravelly Sand Medium brown, dry, no odour.	1		13		
1.0					14		
1.5					15		
2.0					9		
2.5			2				
3.0	1.0	Saturated @ 2.5 ft, no recovery	3		2		
3.5					0		
4.0					0		
4.5			4		0		
5.0		Clayey Sand Brown & black, saturated, unusual odour.	5				
5.5					16		
6.0					6		
6.5	2.0				8		
7.0			6		14		
8.0			7		13		
8.5					17		
9.0					20		
9.5			8		24		
10.0	3.0	End of Log					

Notes:
- No monitoring well installed.

A & A Environmental Consultant Inc.
16 Young Street Woodstock, ON

LOGGED BY: M. Richardson	**COMPLETION DEPTH:** 10 feet
REVIEWED BY: A. Rasoul	**DRILL METHOD:** Split spoon
DRILL DATE: Oct 1, 2019	**Page:** 1 of 1

APPENDIX C – Grain Size Distribution and Test Results

Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

Aug 2nd, 2019

File No. 7-18-0032-57
Stoney Creek Office

A&A Environmental Consultants
16 Young Street
Woodstock, Ontario
N4S 3L4

Attention: Mr. Thomas Demers

RE: LABORATORY TEST RESULTS
PROJECT – 4545 BVD Caledon

Dear Sir:

This report presents the results of laboratory testing carried out on a soil sample received at our Stoney Creek laboratory on July 18th, 2019.

The laboratory testing included the following:

- Particle size analyses per ASTM D422 & D2217;
- Atterburg Limits per ASTM 4318;

The results of the testing are summarized in the attached Table 1 and shown on the accompanying Figures.

We trust this information is sufficient for your present purposes. Should you have any questions concerning the above, please do not hesitate to contact the undersigned.

Yours truly,
Terraprobe Inc.

Aaron Skipper

Aaron Skipper
Lab Supervisor

Rev: P. Cannon, P. Eng.

Terraprobe Inc.

Greater Toronto	Hamilton – Niagara	Central Ontario	Northern Ontario
11 Indell Lane	903 Barton Street, Unit 22	220 Bayview Drive, Unit 25	1012 Kelly Lake Rd., Unit 1
Brampton, Ontario L6T 3Y3	**Stoney Creek**, Ontario L8E 5P5	**Barrie**, Ontario L4N 4Y8	**Sudbury**, Ontario P3E 5P4
(905) 796-2650 Fax: 796-2250	(905) 643-7560 Fax: 643-7559	(705) 739-8355 Fax: 739-8369	(705) 670-0460 Fax: 670-0558

www.terraprobe.ca

Table 1

Summary of Laboratory Testing

A & A Project No. 4545 BVD Caledon

Sample No.	Depth (ft.)	Soil Description	Atterburg Limits (%)			Particle Size Distribution (Figure No.)
			w_L	w_P	I_P	
BH 1	10 – 12’	Silty Clay some Sand and Gravel	28.75	15.11	13.64	1
BH 2	12.5 – 14.5’	Clayey Silt with Sand trace Gravel	30.50	15.84	14.66	2
BH 3	5 – 7	Clayey Silt with Sand trace Gravel	30.39	16.91	13.48	3
BH 4	7.5 – 9.5’	Clayey Silt with Sand trace Gravel	29.75	15.38	14.37	4
BH 5	2.5 – 4.5’	Silt and Clay some Sand trace Gravel	33.01	15.99	17.02	5
BH 6	7.5 – 9.5’	Silt and Clay some Sand trace Gravel	29.74	16.19	13.55	6

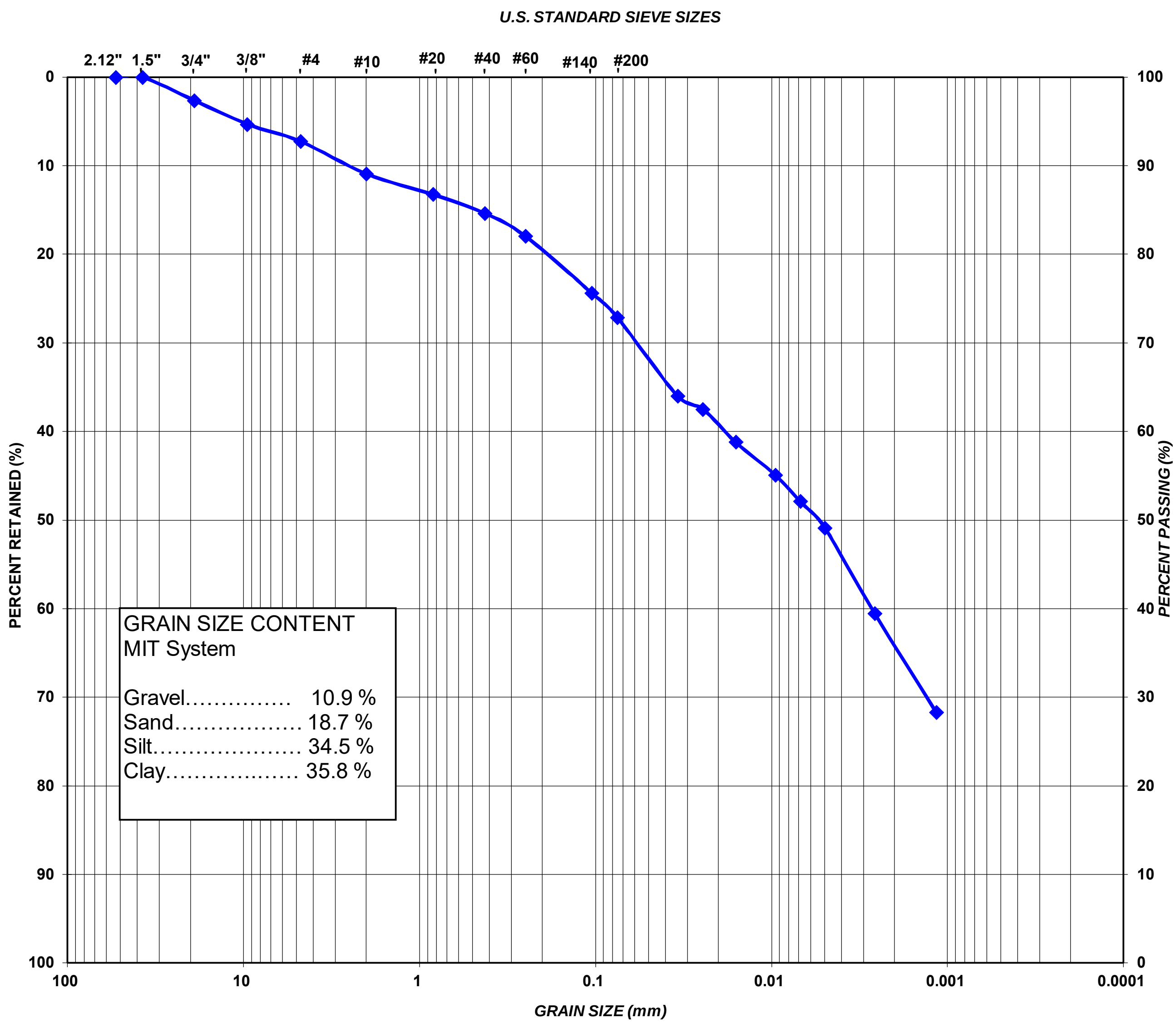
Notes: To be read with accompanying letter.

SIEVE AND HYDROMETER ANALYSIS TEST REPORT

PROJECT: **4545 BVD Caledon**
LOCATION: **Woodstock, Ontario**
CLIENT: **A&A Enviromental**
BOREHOLE: **1**
SAMPLE NUMBER:
SAMPLE DEPTH: **10 - 12'**
SAMPLE DESCRIPTION: **Silty Clay some Sand and Gravel**

FILE NO.: **7-18-0032-57**
LAB NO.: **S3411**
SAMPLE DATE: **July 18, 2019**
SAMPLED BY: **Client**

GRAIN SIZE DISTRIBUTION



MIT SYSTEM	GRAVEL	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT	CLAY

UNIFIED SYSTEM	GRAVEL: COARSE	GRAVEL: FINE	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT AND CLAY

Figure 1

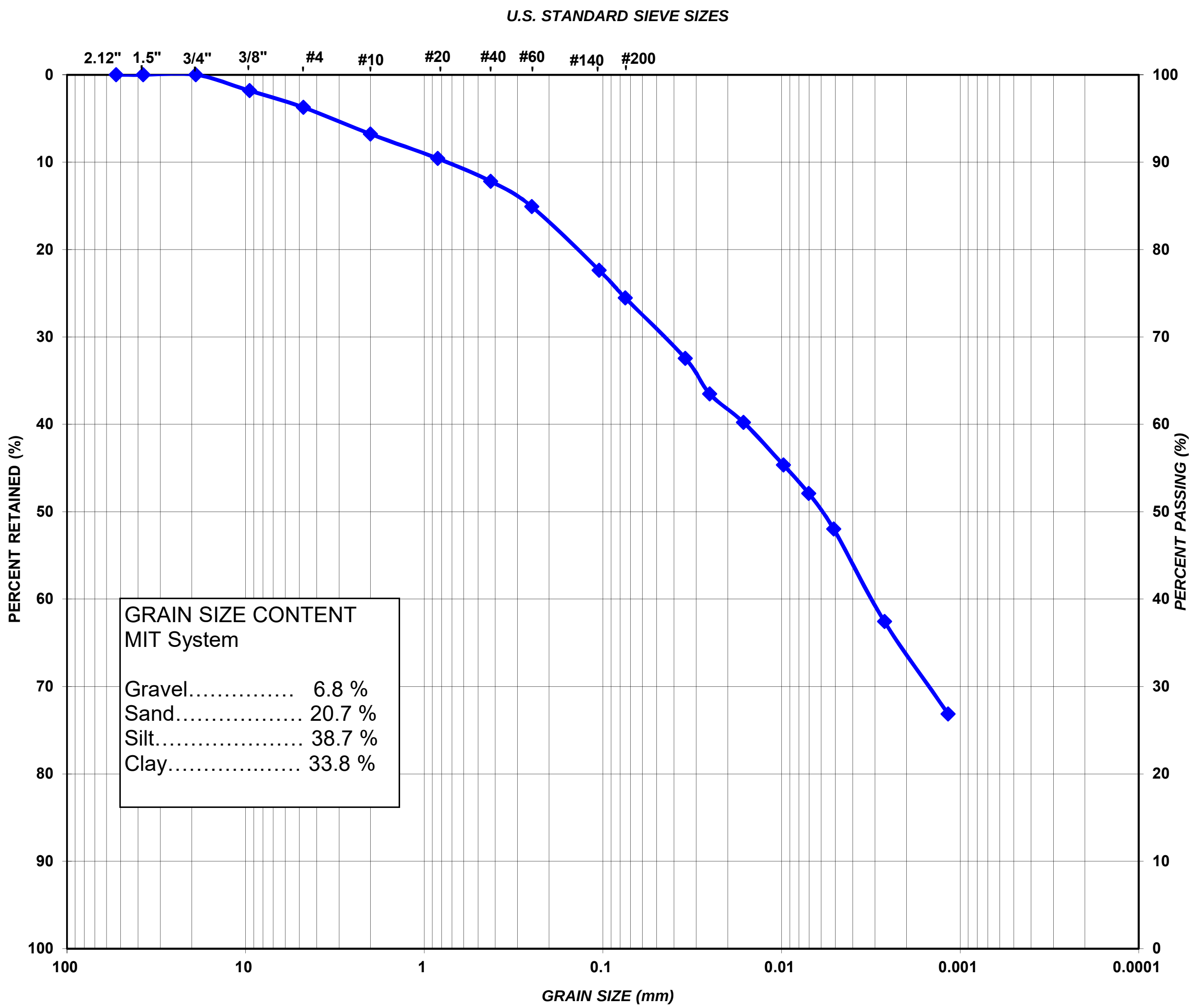
Terraprobe

SIEVE AND HYDROMETER ANALYSIS TEST REPORT

PROJECT: **4545 BVD Caledon**
LOCATION: **Woodstock, Ontario**
CLIENT: **A&A Enviromental**
BOREHOLE: **2**
SAMPLE NUMBER:
SAMPLE DEPTH: **12.5 - 14.5'**
SAMPLE DESCRIPTION: **Clayey Silt with Sand trace Gravel**

FILE NO.: **7-18-0032-57**
LAB NO.: **S3412**
SAMPLE DATE: **July 18, 2019**
SAMPLED BY: **Client**

GRAIN SIZE DISTRIBUTION



MIT SYSTEM	GRAVEL	SAND			SILT	CLAY
		COARSE	MEDIUM	FINE		

UNIFIED SYSTEM	GRAVEL		SAND			SILT AND CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Figure 2

Terraprobe

SIEVE AND HYDROMETER ANALYSIS TEST REPORT

PROJECT: **4545 BVD Caledon**
LOCATION: **Woodstock, Ontario**
CLIENT: **A&A Enviromental**
BOREHOLE: **3**
SAMPLE NUMBER:
SAMPLE DEPTH: **5 - 7'**
SAMPLE DESCRIPTION: **Clayey Silt with Sand trace Gravel**

FILE NO.: **7-18-0032-57**
LAB NO.: **S3413**
SAMPLE DATE: **July 18, 2019**
SAMPLED BY: **Client**

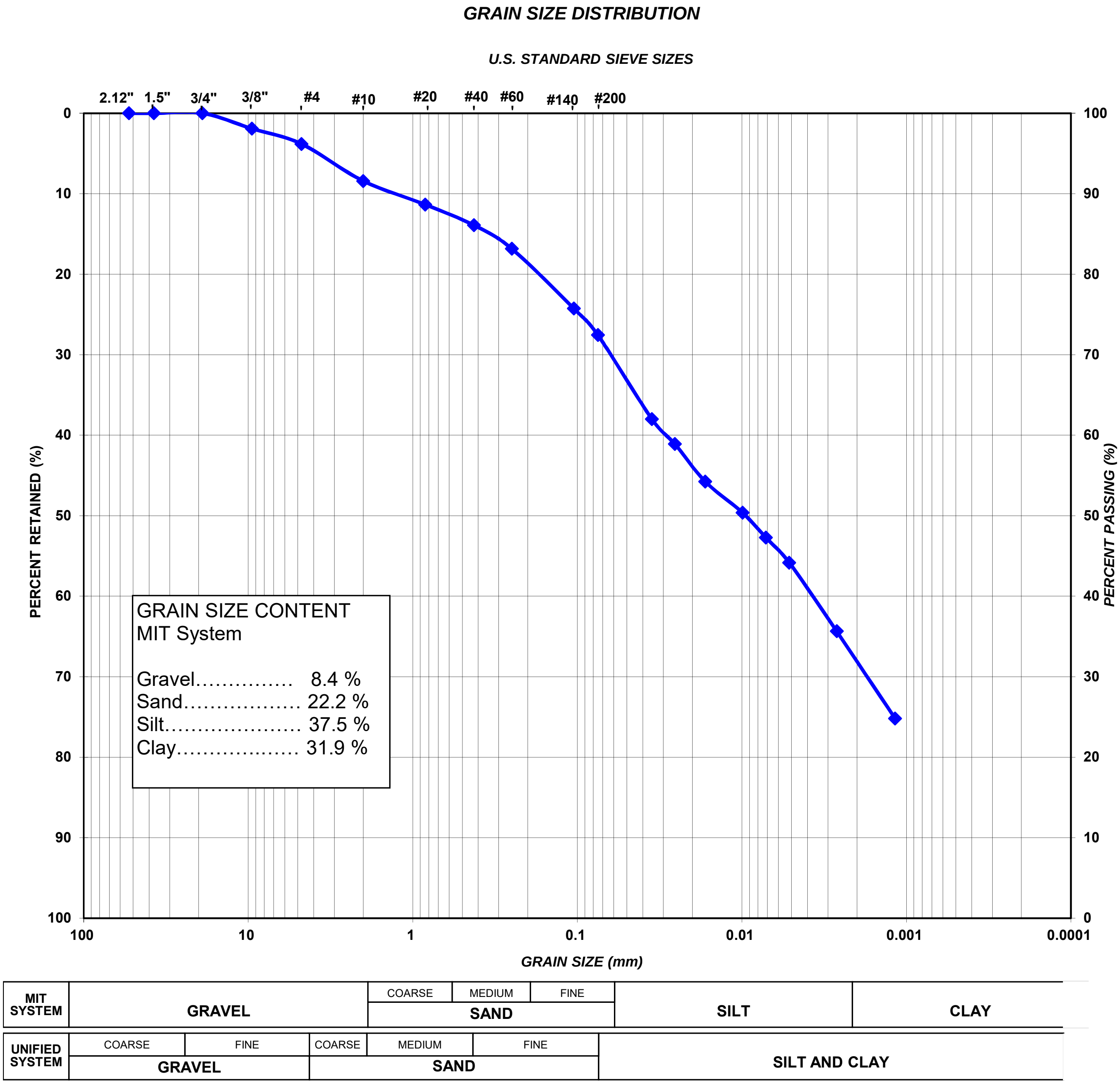


Figure 3

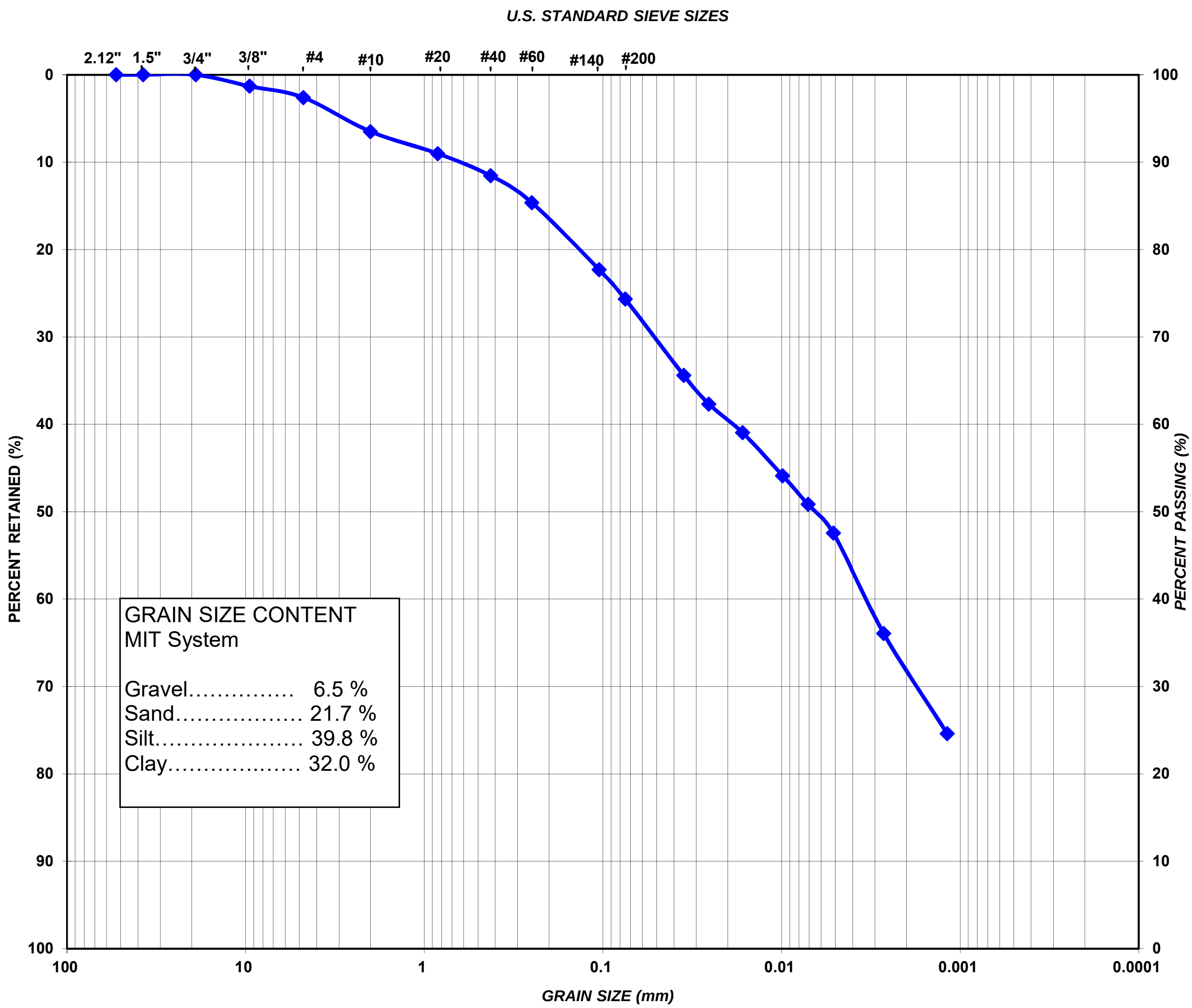
Terraprobe

SIEVE AND HYDROMETER ANALYSIS TEST REPORT

PROJECT: **4545 BVD Caledon**
LOCATION: **Woodstock, Ontario**
CLIENT: **A&A Enviromental**
BOREHOLE: **4**
SAMPLE NUMBER:
SAMPLE DEPTH: **7.5 - 9.5'**
SAMPLE DESCRIPTION: **Clayey Silt with Sand trace Gravel**

FILE NO.: **7-18-0032-57**
LAB NO.: **S3414**
SAMPLE DATE: **July 18, 2019**
SAMPLED BY: **Client**

GRAIN SIZE DISTRIBUTION



MIT SYSTEM	GRAVEL	SAND – COARSE	SAND – MEDIUM	SAND – FINE	SILT	CLAY

UNIFIED SYSTEM	GRAVEL – COARSE	GRAVEL – FINE	SAND – COARSE	SAND – MEDIUM	SAND – FINE	SILT AND CLAY

Figure 4

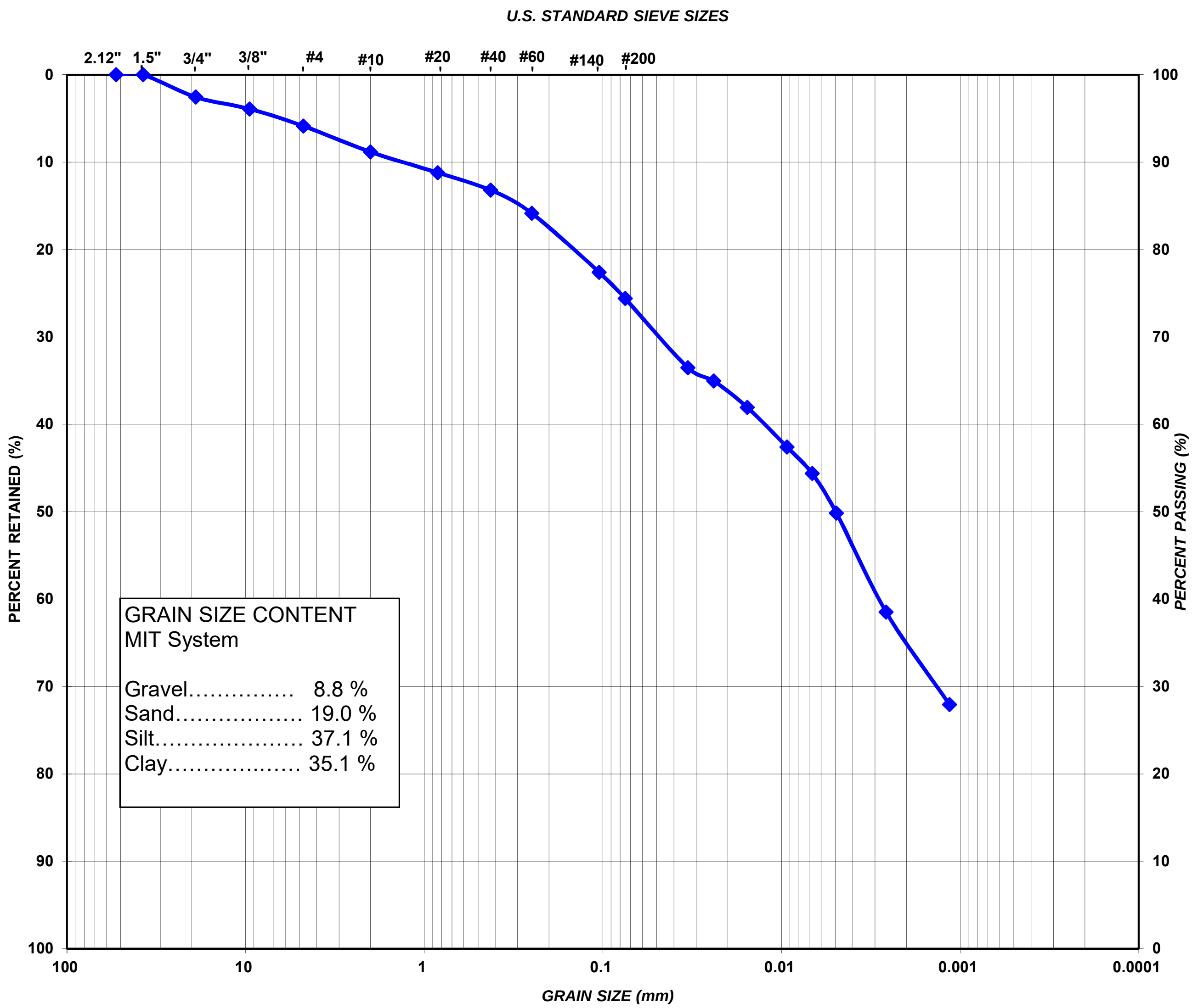
Terraprobe

SIEVE AND HYDROMETER ANALYSIS TEST REPORT

PROJECT: **4545 BVD Caledon**
LOCATION: **Woodstock, Ontario**
CLIENT: **A&A Enviromental**
BOREHOLE: **5**
SAMPLE NUMBER:
SAMPLE DEPTH: **2.5 - 4.5'**
SAMPLE DESCRIPTION: **Silt and Clay some Sand trace Gravel**

FILE NO.: **7-18-0032-57**
LAB NO.: **S3415**
SAMPLE DATE: **July 18, 2019**
SAMPLED BY: **Client**

GRAIN SIZE DISTRIBUTION



MIT SYSTEM	GRAVEL	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT	CLAY

UNIFIED SYSTEM	GRAVEL: COARSE	GRAVEL: FINE	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT AND CLAY

Figure 5

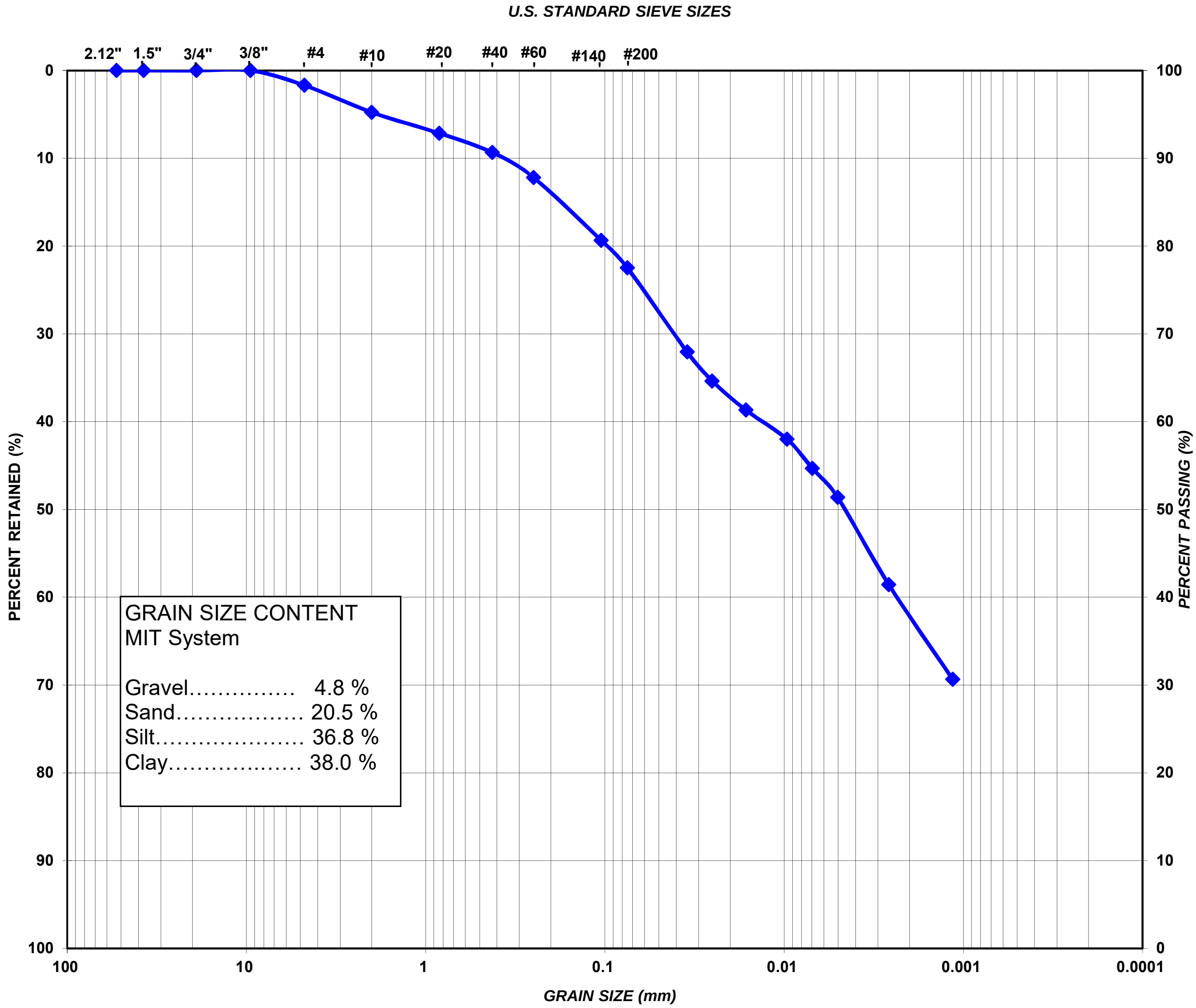
Terraprobe

SIEVE AND HYDROMETER ANALYSIS TEST REPORT

PROJECT: **4545 BVD Caledon**
LOCATION: **Woodstock, Ontario**
CLIENT: **A&A Enviromental**
BOREHOLE: **6**
SAMPLE NUMBER:
SAMPLE DEPTH: **7.5 - 9.5'**
SAMPLE DESCRIPTION: **Silt and Clay some Sand trace Gravel**

FILE NO.: **7-18-0032-57**
LAB NO.: **S3416**
SAMPLE DATE: **July 18, 2019**
SAMPLED BY: **Client**

GRAIN SIZE DISTRIBUTION



MIT SYSTEM	GRAVEL	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT	CLAY

UNIFIED SYSTEM	GRAVEL: COARSE	GRAVEL: FINE	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT AND CLAY

Figure 6

Oct 15th, 2019

File No. 7-18-0032-65
Stoney Creek Office

A&A Environmental Consultants
16 Young Street
Woodstock, Ontario
N4S 3L4

Attention: Mr. Thomas Demers

RE: LABORATORY TEST RESULTS PROJECT – 4545 BVD Caledon

Dear Sir:

This report presents the results of laboratory testing carried out on soil samples received at our Stoney Creek laboratory on Oct 4th, 2019.

The laboratory testing included the following:

- Water content per ASTM D2216;
- Particle size analyses per ASTM D422 & D2217;
- Atterburg Limits per ASTM 4318;

The results of the testing are summarized in the attached Table 1 and shown on the accompanying Figures.

We trust this information is sufficient for your present purposes. Should you have any questions concerning the above, please do not hesitate to contact the undersigned.

Yours truly,
Terraprobe Inc.

Aaron Skipper

Aaron Skipper
Lab Supervisor

Rev: P. Cannon, P. Eng.

Table 1

Summary of Laboratory Testing

A & A Project No. 4545 BVD Caledon

Sample No.	Depth (ft.)	Water Content (%)	Soil Description	Atterburg Limits (%)			Particle Size Distribution (Figure No.)
				w_L	w_P	I_P	
BH 8	7.5 – 9.5’	17.3	Sandy Clay with Silt some Gravel	28.44	14.71	13.74	1
BH 7	2.5 – 4.5’	7.4	Sand and Gravel trace Silt and Clay		Non Plastic		2

Notes: To be read with accompanying letter.

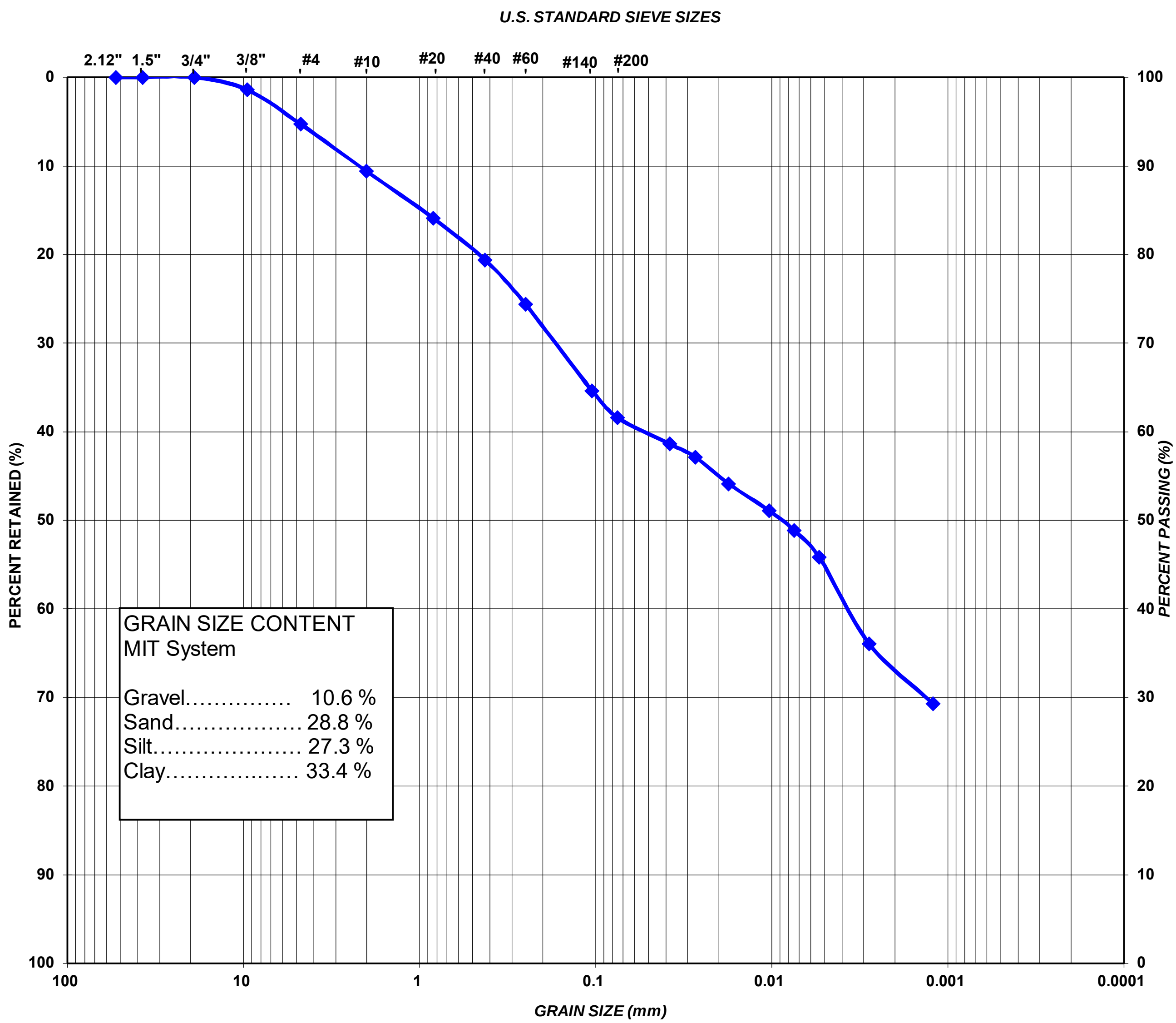
Terraprobe

SIEVE AND HYDROMETER ANALYSIS TEST REPORT

PROJECT: **4545 BVD Caledon**
LOCATION: **Woodstock, Ontario**
CLIENT: **A&A Enviromental**
BOREHOLE: **8**
SAMPLE NUMBER:
SAMPLE DEPTH: **7.5 - 9.5'**
SAMPLE DESCRIPTION: **Sandy Clay with Silt some Gravel**

FILE NO.: **7-18-0032-65**
LAB NO.: **S3567**
SAMPLE DATE: **Oct 4, 2019**
SAMPLED BY: **Client**

GRAIN SIZE DISTRIBUTION



MIT SYSTEM	GRAVEL	SAND (COARSE / MEDIUM / FINE)	SILT	CLAY

UNIFIED SYSTEM	GRAVEL (COARSE / FINE)	SAND (COARSE / MEDIUM / FINE)	SILT AND CLAY

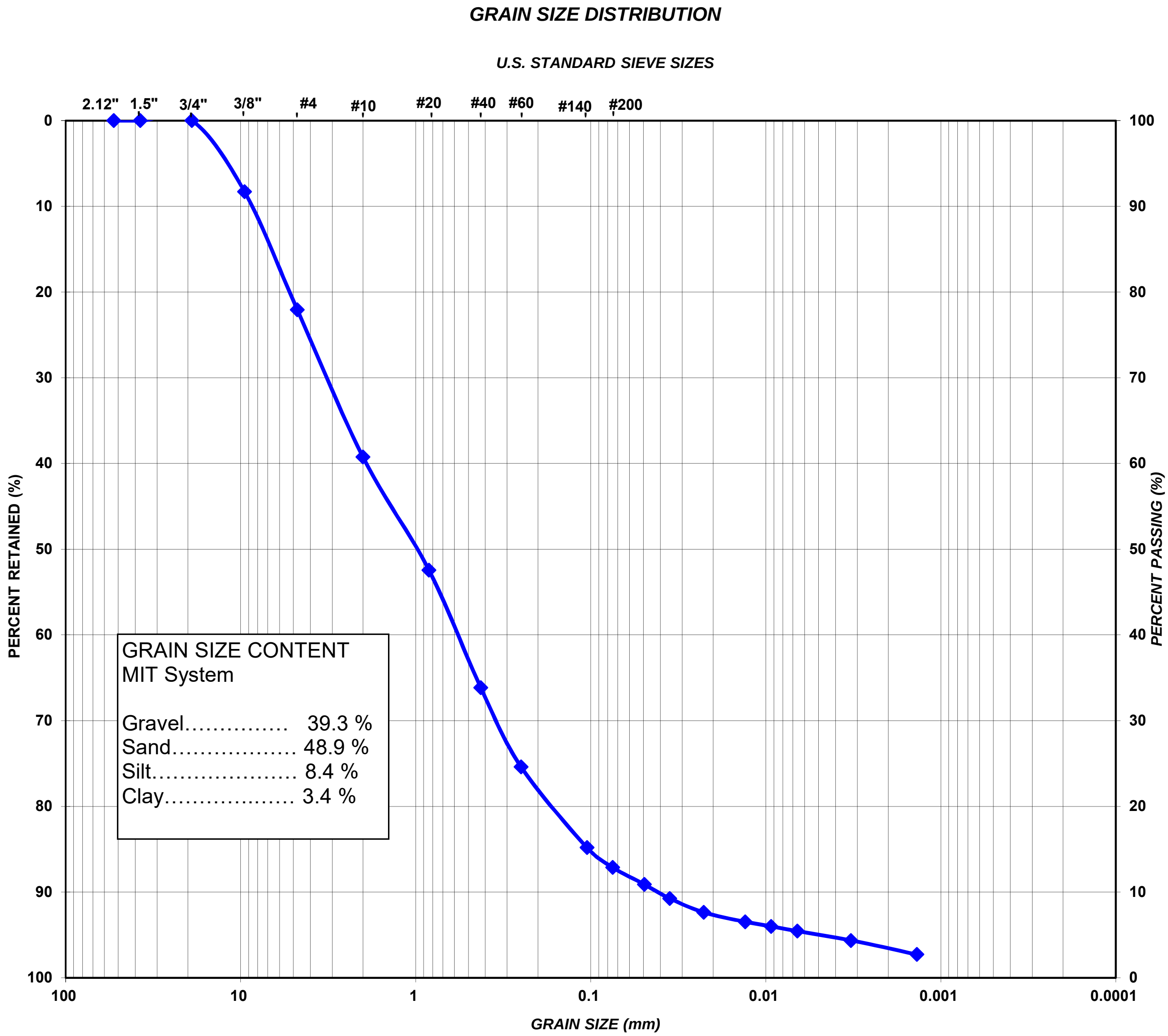
Figure 1

Terraprobe

SIEVE AND HYDROMETER ANALYSIS TEST REPORT

PROJECT: **4545 BVD Caledon**
LOCATION: **Woodstock, Ontario**
CLIENT: **A&A Enviromental**
BOREHOLE: **7**
SAMPLE NUMBER:
SAMPLE DEPTH: **2.5 - 4.5'**
SAMPLE DESCRIPTION: **Sand and Gravel trace Silt and Clay**

FILE NO.: **7-18-0032-65**
LAB NO.: **S3568**
SAMPLE DATE: **Oct 4, 2019**
SAMPLED BY: **Client**



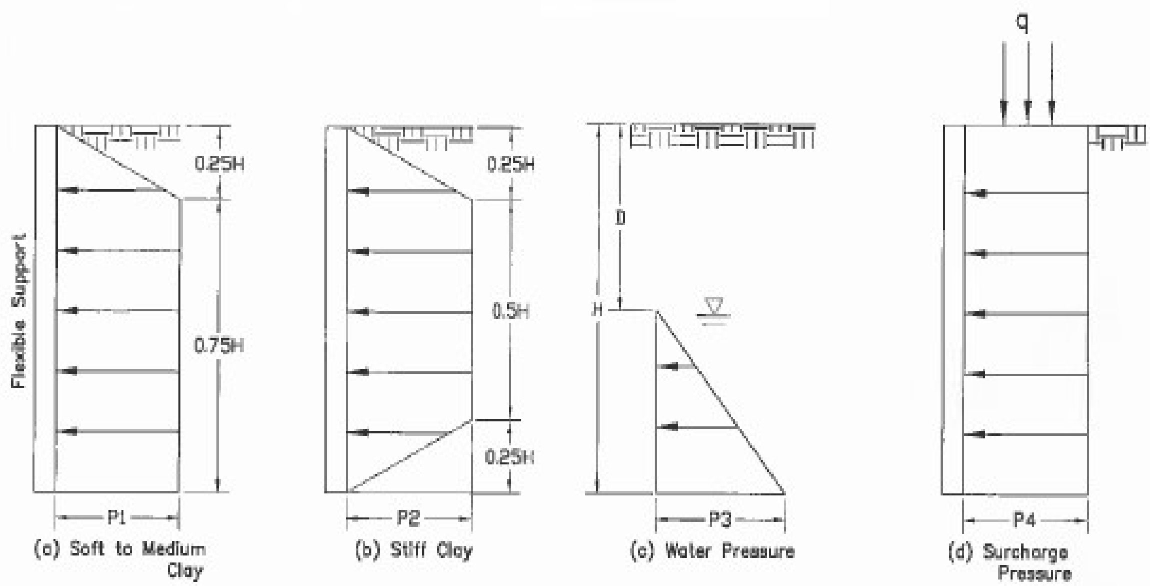
MIT SYSTEM	GRAVEL	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT	CLAY

UNIFIED SYSTEM	GRAVEL: COARSE	GRAVEL: FINE	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT AND CLAY

Figure 2

APPENDIX D – Diagrams and Figures

Figure 6 – Lateral Pressure Diagrams for Open Cuts in Cohesive Soils



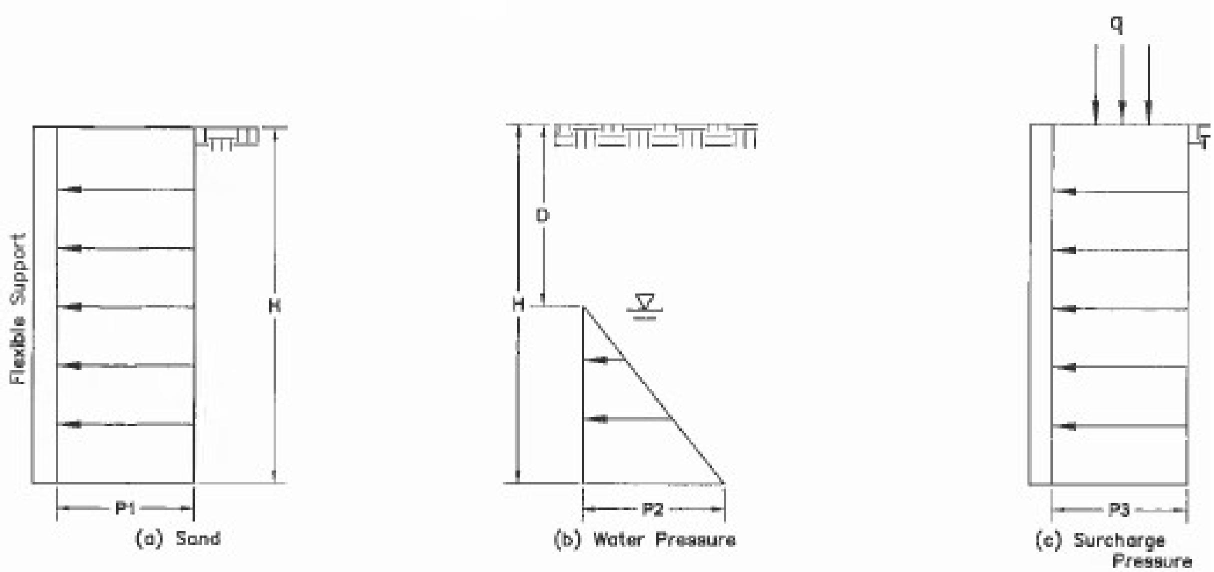
Empirical Pressure Distributions

Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $\gamma H - 4C$, psf
P2 = Lateral earth pressure = $0.4\gamma H$, psf
P3 = Water pressure = γ_w (H-D), psf
P4 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
C = Drained shear strength or cohesion, psf
K_a = Coefficient of active earth pressure

Notes:
1. All pressures are additive.
2. No safety factors are included.
3. For use only during long term construction.
4. If $\gamma H/C < 4$, use section (b),
   If $4 < \gamma H/C < 6$, use larger of section (a) or (b),
   If $\gamma H/C > 6$, use section (a).

Reference: Peck, R.B. (1969), "Deep Excavation and Tunneling in soft Ground", 7th ICSMFE, State of art volume, pp. 225-290.

Figure 7 – Lateral Pressure Diagram for Open Cuts in Sand



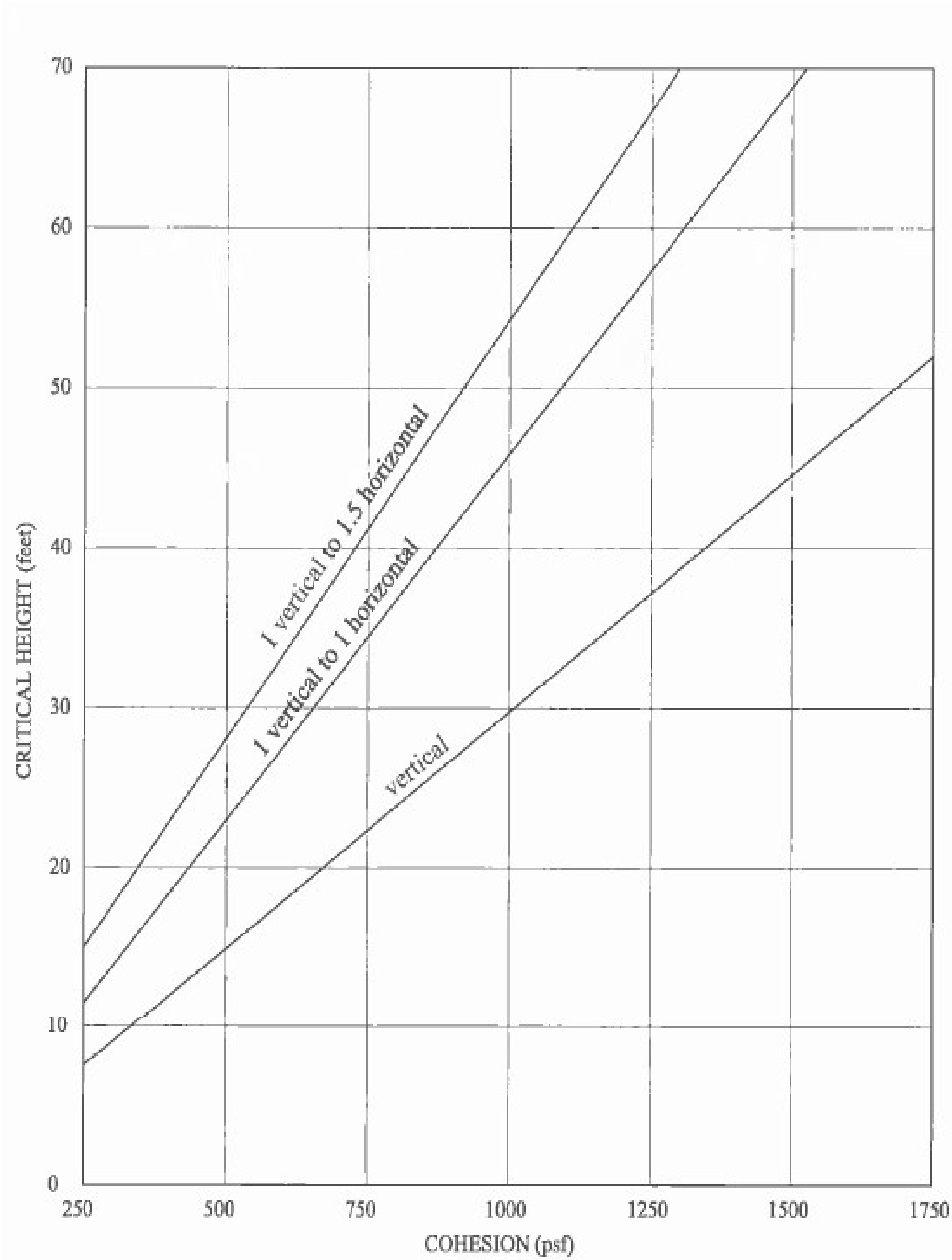
Empirical Pressure Distributions

Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $0.65^{*}\gamma HK_a$, psf
P2 = Water pressure = γ_w (H-D), psf
P3 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
K_a = Coefficient of active earth pressure = $(1-\sin\varphi)/(1+\sin\varphi)$
φ = Drained friction angle

Notes:
1. All pressures are additive.
2. No safety factors are included.

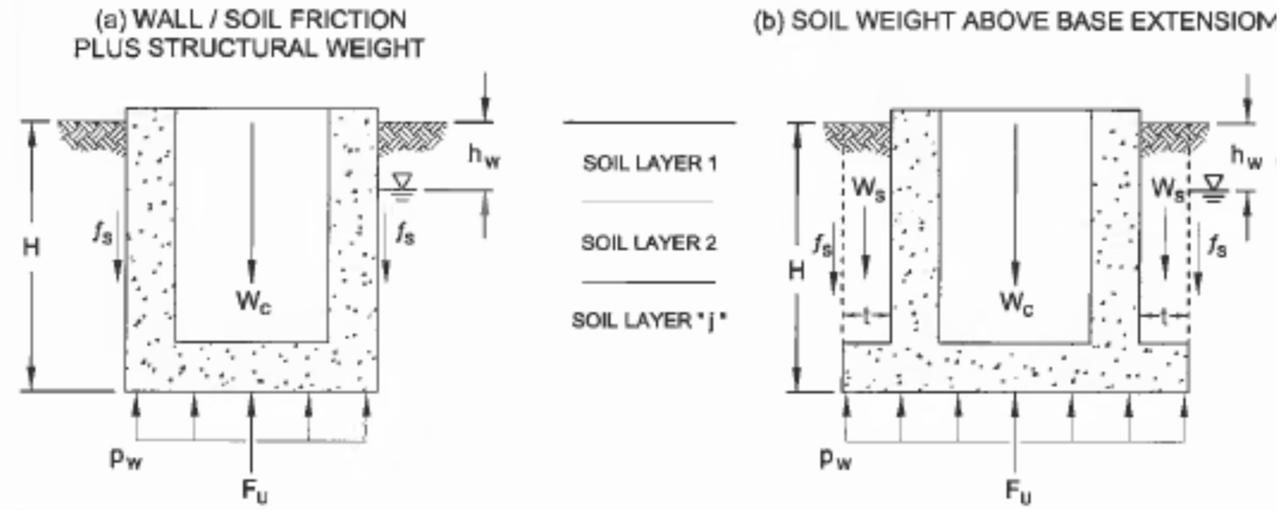
Reference: Peck, R.B. (1969), "Deep Excavation and Tunneling in soft Ground", 7th ICSMFE, State of art volume, pp. 225-290.

Figure 8 – Critical Heights of Cut Slopes in Non-Fissured Clays



A & A ENVIRONMENTAL CONSULTANTS INC.

Figure 9 – Uplift Pressure for Buried Structure



(a)

cohesive soils: $f_{s_j} = \alpha\ c_j \leq 3{,}000$ psf

cohesionless soils: $f_{s_j} = 0.75\ K_s\ \sigma_{v_j} \tan\delta_j$

$$Q_s = P_s \sum f_{s_j} h_j$$

$$\frac{W_c}{S_{f_a}} + \frac{Q_s}{S_{f_b}} \geq F_u$$

(b)

cohesive soils: $f_{s_j} = c_j \leq 3{,}000$ psf

cohesionless soils: $f_{s_j} = 0.75\ K_s\ \sigma_{v_j} \tan\Phi_j$

$$Q_s = P_s \sum f_{s_j} h_j$$

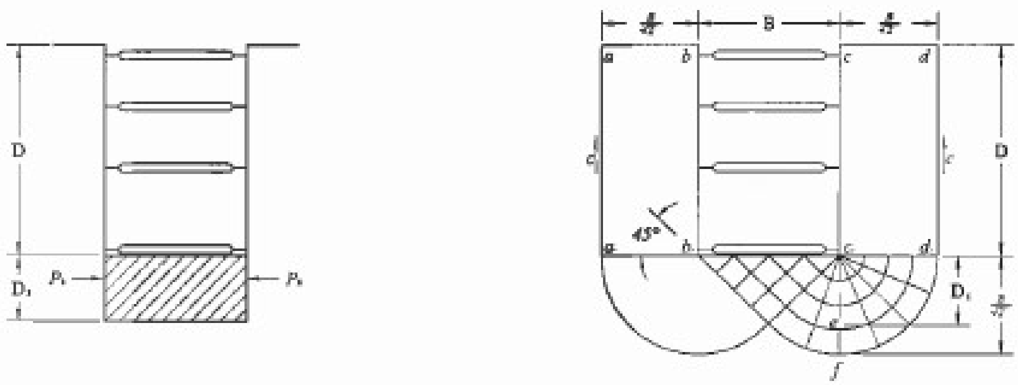
$$\frac{W_c}{S_{f_a}} + \frac{Q_s}{S_{f_b}} + \frac{W_s}{S_{f_c}} \geq F_u$$

Where:

- A_B = area of base, sq. ft.
- H = buried height of structure, ft.
- h_w = depth to water table, ft.
- p_w = $\gamma_w(H - h_w)$, unit hydrostatic uplift, psf.
- γ_w = 62.4 pcf, unit weight of water
- F_u = $p_w A_B$, hydrostatic uplift force, lbs.
- f_{s_j} = unit frictional resistance of soil layer "j", psf.
- c_j = undrained cohesion of soil layer "j", psf.
- α = 0.55, cohesion factor between soil and structure wall
- σ_{v_j} = effective overburden pressure at midpoint of soil layer "j", psf.
- δ_j = 0.75 Φ_j, friction angle between soil layer "j" and concrete wall, degrees
- Φ_j = internal angle of friction of soil layer "j", degrees
- K_s = 0.4, coefficient of lateral pressure
- h_j = thickness of soil layer "j", ft.
- j = 1, 2, ...
- P_s = perimeter of structure base, ft.
- Q_s = ultimate skin friction, lbs.
- W_c = weight of structure, lbs.
- W_s = weight of backfill above base extension, lbs.
- S_{f_a} = 1.1, factor of safety for dead weight of structure
- S_{f_b} = 3.0, factor of safety for soil / structure friction
- S_{f_c} = 1.5, factor of safety for soil weight above base extension
- t = width of base extension, ft.

NOTE: neglect f_s in upper 5 feet for expansive clay with a plasticity index > 20.

Figure 10 – Bottom Stability for Braced Excavation in Clay



Factor of Safety against bottom of heave,

$$F.S = \frac{NcC}{(\gamma D + q)}$$

where, Nc = Coefficient depending on the dimension of the excavation (see Figure at the bottom)
C = Undrained shear strength of soil in zone immediately around the bottom of the excavation,
γ = Unit weight of soil,
D = Depth of excavation,
q = Surface surcharge.

If F.S < 1.5, sheeting should be extended further down to achieve stability

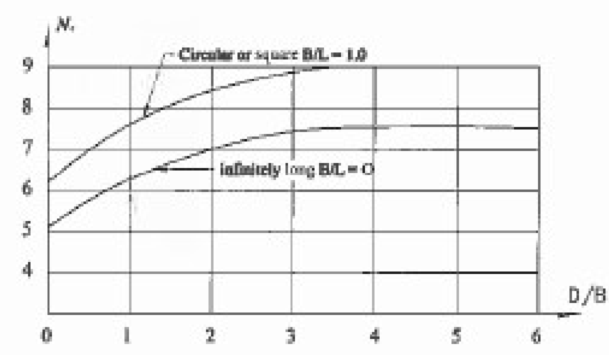
$$\text{Depth of Buried Length, } (D_1) = \frac{1.5(\gamma D+q)-NcC}{(C/B)-0.5\gamma}\ ;\ D_1 \geq 5\ ft.$$

Pressure on buried length, P_1

For $D_1 < 0.47B$; $P_1 = 1.5\, D_1(\gamma D - 1.4\ CD/B - 3.14C)$

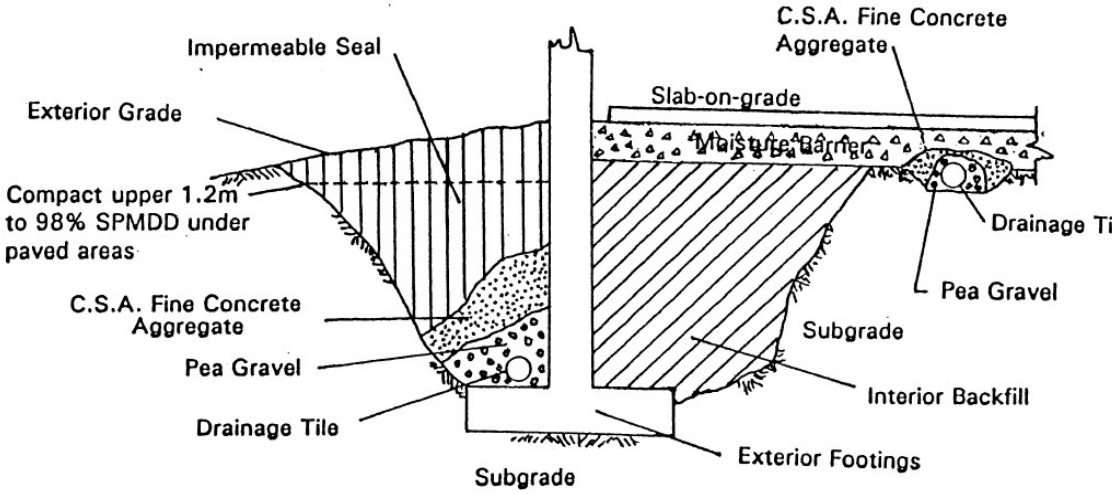
For $D_1 > 0.47B$; $P_1 = 0.7\ (\gamma DB - 1.4\ CD - 3.14CB)$

where; B = width of excavation



N_c rectangular = $(0.84 + 0.16B/L)N_c$ square

Figure 11 – Drainage and Backfilling Recommendations (not to scale)



NOTES:

1. Drainage tile to consist of 10cm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet. Invert to be minimum of 15cm (6") below underside of floor slab.
2. Pea gravel 15cm (6") top and sides of drain. If drain is not on footing, place 10cm (4") of pea gravel below drain. 20mm (3/4") stone is an alternative, provided it is covered by an approved geotextile.
3. C.S.A. fine concrete aggregate to act as filter material. Minimum 30cm (12") top and side of tile drain. This may be replaced by an approved porous plastic membrane as indicated in 2.
4. Impermeable backfill seal-compacted clay, clay silt or equivalent. If original soil is free-draining, seal may be omitted.
5. The interior fill may be any clean, non organic soil which may be compacted to at least 98% Standard Proctor density in this confined space.
6. Do not use heavy compaction equipment within 0.5m (18") of the wall. Do not fill or compact within 1.8m (6') of wall unless the fill is placed on both sides simultaneously.
7. Moisture barrier to be at least 20cm (8") of compacted Granular "A" fill or equivalent free-draining material to be approved by our geotechnical staff.
8. The moisture barrier is to be compacted to 98% Standard Proctor maximum dry density.
9. Slab-on-grade should not be structurally connected to the wall or the footing.
10. Exterior grade to slope away from wall.
11. Underfloor drain invert to be at least 300mm (1') below the underside of floor slab. Tile placed in parallel rows 6-8m (20'- 25') centres one way.
12. Do not connect the underfloor drains to perimeter drains.
13. If the 20mm (3/4") stone requires surface blinding, use 6mm (1/4") stone chips.

DRAINAGE AND BACKFILL RECOMMENDATIONS
Not to Scale