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Geotechnical Engineering Report
Residential Property Located at
14245 Highway 50, Caledon, Ontario

Report #6069 – Columbia Square Inc. Caledon
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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1.0 INTRODUCTION

1.1 Proposed Construction

Columbia Square Inc. (the Client), retained the services of A & A Environmental Consultants Inc. (A&A) to conduct a geotechnical investigation for a proposed development on a property located at 14245 Highway 50, Caledon, Ontario. Ten 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:

- The purpose of the geotechnical investigation will be to explore the subsurface soil and groundwater conditions at the subject site. A total of ten boreholes will be drilled to cover the area of the proposed development to a maximum depth of 18.3 metres. Borehole logs will be recorded showing the soil types, groundwater condition, and horizons.
- Select samples obtained from the field investigation will be tested in the laboratory to determine soil properties essential to the preparation of the report. It is essential that the natural moisture content of samples be determined at the time of the investigation. Classification testing of select samples will carried out on soil samples including grain size analysis, Atterberg limits, moisture content determinations, etc. will be carried out in accordance with recognized practice;
- Provide a reasonable conclusion regarding the soil properties of the site;
- Based on the findings of the investigation, geotechnical design, and construction recommendations will to be provided for the building.

3.0 SITE DESCRIPTION

3.1 Current Land Use and Location

The site is rectangular shaped lot located at the north corner of the Highway 50 and Columbia Way intersection with an approximate UTM coordinates of Zone 17T; 600066 m Easting and 4860703 m Northing (Figure 1, Appendix A). The property is currently used as agricultural land with no building. Lands adjacent to the subject site consist of residential to the east, institutional to the north, and commercial to the west and south.

3.2 Topography and Drainage

The topography of the subject site was observed to be generally flat with a slight southeastern slope toward Columbia Way. According to the topographic map acquired from Natural Resources Canada website the elevation of the subject site ranges from approximately 263 to 266 meters above sea level (masl) (Figure2, Appendix A). The topography in the vicinity of the subject site (a 100-meter radius) ranges from approximately 266 masl to the north to 262 masl to the south. The Humber River is located on south of the subject site beyond Highway 50. The surface water is expected to infiltrate the permeable ground surface.

3.3 Geology

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 “Paleozoic Geology of Southern Ontario” identifies the site to be within the Georgian Bay Formation; characterized by shale and limestone.
- The “Physiography of Southern Ontario” identifies the site on the till plains (drumlinized) landform in the South Slope region.

- The “Quaternary Geology” identifies the majority of the site as Halton till consisting of predominantly silt to silty clay matrix; which is high in matrix carbonate content, clast poor, and Pleistocene.
- The “Surficial Geology” identifies the site as Clay to silt-textured till deposits derived from glaciolacustrine deposits or shale.
- The "Bedrock Geology of Ontario" identified the site to be part of the Georgian Bay Formation; Blue Mountain Formation; Billings Formation; Collingwood Member and Eastview Member, characterized by shale, limestone, dolostone and siltstone.

4.0 PROPOSED DEVELOPMENT

It is understood that the proposed future development will consist of the following:

- Three 3-storey back-to-back townhouse buildings (Phase 1A) each consisting of 8 townhouse units with no underground parking.
- Six 3-storey back-to-back townhouse buildings (Phase 1B) each consisting of 18 or 20 townhouse units. There will be one level of underground parking under each townhouse unit.
- Two mixed-used 8-storey buildings with a podium and one level of underground parking, (Phase 2) each consisting of 117 units, with a total gross floor area of 24,312 m^2. There will be a total number of 349 underground parking spaces and 122 surface level parking spaces.
- One 8-storey retirement residence building (Phase 3) consisting of 159 units. There will be two levels of underground parking under the retirement residence building, with additional visitor parking spaces around the building.
- There will be two access points to this site, one off of Highway 50 and one off of Columbia Way.

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:

- Between May 25, 2021 and June 23, 2021, A&A attended the site located at 14245 Highway 50, Caledon, Ontario.
- Boreholes BH1, BH2, BH3, and BH4 were advanced by Aardvark Drilling Inc., using a CME 850 Truck mounted drill rig. 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.
- Boreholes BH 5, BH6, BH7, BH8, BH9, and BH10 were advanced by A&A using a Geoprobe 7730 track mounted drill rig. 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.
- Table 1 indicates the depth and location for each borehole advanced. Figure 5 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. Five samples were selected for geotechnical analysis as the lithology and soil conditions across the site were fairly uniform.
- All boreholes were used for the geotechnical investigation. BH1, BH4, BH5, and BH7 had monitoring wells installed during the investigation. These wells were used as part of the hydrogeological investigation and to determine the water table elevation for this investigation. All boreholes except BH1, BH4, BH5, and BH7 were refilled with low-permeability bentonite pellets.

Table 1 – Borehole Advanced Depths and Location

Borehole	Location	Depth (m)	Elevation (masl)
BH/MW1	Central West Boundary	18.29	264
BH2	West Portion of Southwest boundary	18.29	263
BH3	Central East Boundary	15.24	264
BH/MW4	Northwest of Sidewalk	15.24	265
BH/MW5	Near Northwest Boundary of Site	7.62	265
BH6	South Corner of Site	7.62	263
BH/MW7	Northeast of HWY 50 Entrance	7.62	266
BH8	North of HWY 50 Entrance	7.62	264
BH9	central North Portion of Site	4.57	265
BH10	central North Portion of Site	3.05	264

5.2 Sampling Procedures

Select samples recovered from the geotechnical investigation were submitted to Orbit Engineering Inc. (Orbit), 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 logs in Appendix B. The results of the grain size distribution tests are also shown on the borehole logs (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 logs are also included in Appendix B.

Select bagged samples taken from the boreholes were analyzed at Orbit 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 drilling program for this study indicates that the overburden deposits are consistent across boreholes at the approximate proposed foundations depth of 1.2 to 3.0 mbgl. BH1 to BH10 have a silt and clay mixture at the foundation depth. All boreholes were terminated due to either refusal or being below the intended foundations of the proposed development.

The combination of lab results and standard penetration test N values (blows/foot) were then used to estimate geotechnical resistance 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

All ten boreholes revealed underlain the surface to be characterised as follows:

- **Topsoil**

  All of the boreholes encountered a layer of topsoil at the ground surface. The thickness of the topsoil layer ranged from approximately 0 – 10 cm. The top soil was light brown in colour, dry and had no odour in any of the boreholes.

- **Silt and Clay**

  After the top soil, a layer of Silt and Clay with trace Gravel and/or Sand spreads across the site at an approximate depth of 0.1 mbgl to 13.5 mbgl. The content of sand material was less that 6% except BH7 where 36% sand was found at a depth of approximately 6.9 to 7.5 mbgl. The clay content of this layer varies between 8 to 44%. Boreholes BH5 to BH10 were terminated within this deposit. This deposit was medium brown to grey in colour in the shallower areas. The water content of test samples from this deposit varies from 14.3% to 28.7%, where water content increased with depth. The SPT-N values vary from 1 to greater than 100 within this layer. The SPT-N values within the major stressing zone for shallow foundation vary within a wide range from 7 to >100. Based on the laboratory and in-situ test results, the soil of this composition will behave geotechnically more like a compact to very dense cohesionless soil.

- **Clayey Silt**

  A Clayey Silt layer with some Sand trace Gravel was found at an approximate depth of 13.5 to 17.9 mbgl. Boreholes BH1 to BH4 were terminated within this deposit. This layer was medium grey in colour and had no odour. The SPT-N values vary from 1 to 37 within this layer. Based on the laboratory and in-situ test results, the soil of this composition will behave geotechnically more like a loose to dense cohesionless soil.

6.3 Summary of Subsurface Conditions to Anticipated Depths of Construction

In the following tables (Tables 2–4), 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. The "Notes on Sample Description" preceding the borehole logs are an integral part of and should be read in conjunction with this report.

Table 2 – Typical Values of Moisture Content

BH #	Depth (m)	Soil Description	Water Content (%)
BH1	16.76 – 17.37	Clayey Silt, some Sand trace Gravel	15.2
BH2	9.91– 10.52	Silt, some Clay trace Sand and Gravel	18.9
BH4	12.95 – 13.56	Silt and Clay, trace Sand	28.7
BH7	6.86 – 7.47	Silt and Sand, trace Clay and Gravel	14.3
BH8	3.05 – 3.66	Silt and Clay, trace Sand and Gravel	21.2

Table 3 – Typical Values of Atterburg Limits (%)

BH #	Depth (m)	Soil Description	Atterberg Limits		
			W_L	W_P	I_P
BH1	16.76 – 17.37	Clayey Silt, some Sand trace Gravel	24.4	14.1	10.3
BH2	9.91– 10.52	Silt, some Clay trace Sand and Gravel	20.5	13.3	7.2
BH4	12.95 – 13.56	Silt and Clay, trace Sand	37.7	18.5	19.2
BH7	6.86 – 7.47	Silt and Sand, trace Clay and Gravel	Non-Plastic		
BH8	3.05 – 3.66	Silt and Clay, trace Sand and Gravel	41.2	19.5	21.7

Table 4 – Sieve and Hydrometer Analysis

BH #	Grain Size Content (%)				Depth (m)	Soil Description
	Gravel	Sand	Silt	Clay		
BH1	1	14	64	21	16.76 – 17.37	Clayey Silt, some Sand trace Gravel
BH2	1	2	84	13	9.91– 10.52	Silt, some Clay trace Sand and Gravel
BH4	----	1	57	42	12.95 – 13.56	Silt and Clay, trace Sand
BH7	4	36	52	8	6.86 – 7.47	Silt and Sand, trace Clay and Gravel
BH8	1	6	49	44	3.05 – 3.66	Silt and Clay, trace Sand and Gravel

6.4 Summary of SPT testing

Summary of the SPT test results for the variation of N values with depth is presented in Table 5. Based on the in-situ testing measurements, the cohesionless soil of compact to dense in compactness condition was generally observed within the influence zone of the shallow foundations for the proposed buildings with no or one-storey underground parking.

Table 5 – Variation of N value with Depth

Depth (mbgl)	SPT N-values (blows/300 mm penetration)									
	BH1	BH2	BH3	BH4	BH5	BH6	BH7	BH8	BH9	BH10
0.0 to 0.46	9	12	6	10	4	4	8	9	22	29
0.76 to 1.22	15	16	12	12	8	9	21	22	14	12
1.52 to 1.98	28	21	24	27	18	9	22	21	29	REF
2.29 to 2.74	30	26	26	29	22	17	12	26	REF	REF
3.05 to 3.35	27	9	48	27	18	22	15	23	REF	
3.81 to 4.27	19	10	17	19	10	38	9	12	REF	
4.57 to 5.03	17	12	19	12	10	18	14	16		
5.33 to 5.79	13	10	50	42	7	11	21	24		
6.10 to 6.55	10	6	23	33	9	8	12			
6.86 to 7.16	12	5	16	14	9	17	18			
7.62 to 7.92	10	8	13	22						
8.38 to 8.69	13	9	10	12						
9.14 to 9.45	12	9	9	11						
9.91 to 10.21	13	11	9	10						
10.67 to 10.97	9	8	8	9						
11.43 to 11.73	11	10	7	11						
12.19 to 12.50	9	11	7	5						
12.95 to 13.26	8	17	7	11						
13.72 to 14.02	6	20	5	12						
14.48 to 14.78	18	18	1	17						
15.24 to 15.54	31	23	9							
16.00 to 16.31	24	21								
16.67 to 17.07	28	37								
17.53 to 17.83	32	21								

6.5 Groundwater Conditions

Four groundwater monitoring wells were installed within the annulus of boreholes BH/MW1, BH/MW4, BH/MW5 and BH/MW7 (Figure 4) on site as part of a hydrogeological study. One existing monitoring well was also observed on site and incorporated into the hydrogeological study. Attempts were made on August 26, 2021 to measure the wells for the determination of water level. The water table was found at an approximate depth of 2.7 to 4.3 mbgl (Table 6). Note that seasonal variations in the water table should be anticipated, with higher levels occurring during wet weather conditions and lower levels occurring during dry weather conditions.

Table 6 – Monitoring Well Details

Monitoring Well #	Location	Water Level (mbgl)	Water level (masl)
EMW1	West Corner of the Site	2.4	261.6
BH/MW1	Central West Boundary	3.1	260.9
BH/MW4	Northwest of Sidewalk	4.3	260.7
BH/MW5	Near Northwest Boundary of Site	2.7	262.3
BH/MW7	Northeast of HWY 50 Entrance	3.4	262.6

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 a site that shows consistent sub surface characteristics. Contractors and/or subcontractors bidding on or undertaking the work should, in this light be reasonably assured those 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 Foundation

In accordance with the 2010 National Building Code of Canada (NBCC), the use of Limit States Design (LSD) is required for the design of buildings and their structural components including foundations. The limit states of LSD design are classified into two groups; the Ultimate Limit States (ULS) and the Serviceability Limit States (SLS). The recommended geotechnical resistances for the building foundations are presented for ULS and SLS conditions.

For foundation design this ultimate resistance value is reduced using a Geotechnical Resistance Factor, Φ, which is based on the reliability index of the geotechnical data used to determine the ultimate resistance for the foundation loading case. The resistance factor values presented on Table 7 should be used for foundation design.

Table 7 – Geotechnical Resistance Factors for Foundations

Geotechnical Case	Resistance Factors, Φ
SHALLOW FOUNDATION	
Vertical resistance by semi-empirical analysis and in-situ test data	0.5
Horizontal resistance against sliding (based on friction)	0.8
DEEP FOUNDATIONS (PILES)	
Vertical resistance by semi-empirical analysis and in-situ test data	0.4
Vertical resistance from analysis of dynamic monitoring results	0.5
Vertical resistance from analysis of static load test results	0.6
Uplift resistance by semi-empirical analysis and in-situ test data	0.3
Uplift resistance from analysis of static load test results	0.4
Lateral load resistance	0.5

The values given for Serviceability Limit States (SLS) geotechnical resistances are based on settlement values of less than 25 mm. Total differential settlements within a building should be also less than 19 mm.

Given the conditions encountered in the boreholes BH5 and BH8, the use of conventional spread footing should provide a practical approach for the three-storey buildings with no basement (Phase 1A). The ultimate bearing capacity for a shallow foundation in the soil is calculated for a square footing with a minimum width of $B = 2.0\ m$. The proposed building may be designed for a factored ultimate bearing resistance of 390 kPa at ULS and a bearing resistance of 220 kPa at SLS (assuming 25 mm of settlement). The proposed values for the soil resistance are based on the assumption that the buildings will to be founded on the natural soil at 1.2 mbgl.

Given the conditions encountered in the boreholes BH7 and BH9, the use of conventional spread footing should provide a practical approach for the three-storey townhouse building units with one level underground parking (Phase 1B). The ultimate bearing capacity for a shallow foundation in the soil is calculated for a square footing with a minimum width of $B = 2.0\ m$. The proposed building may be designed for a factored ultimate bearing resistance of 270 kPa at ULS and a bearing resistance of 190 kPa at SLS (assuming 25 mm of settlement). The proposed values for the soil resistance are based on the assumption that the buildings will to be founded on the natural soil at 2.8 to 3.0 mbgl.

Factored geotechnical bearing resistance at ULS is calculated by applying the geotechnical resistance factor of $\Phi = 0.5$ for shallow foundation designs. The un-factored horizontal resistance of the shallow foundations to sliding can be calculated using the following un-factored coefficient of friction:

- 0.30 between new engineered fill consisting of OPSS Granular A or B (Type II) and precast concrete.
- 24 kPa adhesion between precast concrete and the firm to stiff to cohesive soil. In accordance with Table 6, a resistance factor against sliding of $\Phi = 0.8$ should be applied to obtain the resistance at ULS.

Given the conditions encountered in the boreholes BH1, BH2, and BH6, the use of raft foundations should provide a practical approach for the two mixed-use 8-storey buildings with a podium, and one level of underground parking (Phase 2). Because of the relatively large size of the rafts, the bearing capacity failure at ULS condition is too remote to require consideration. The allowable soil pressure for design at SLS condition is estimated to be 190 kPa using the average SPT-N value within the inference zone. A modulus of subgrade reaction of 3.8 MN/m^3 could be used for structural design of the raft foundations. The proposed values for the soil resistance and modulus of subgrade reaction are based on the assumption that the rafts will to be founded on the natural soil at 2.8 to 3.0 mbgl. The values given for SLS geotechnical resistances are based on maximum settlement values of 50 mm. Total differential settlements within a building should also be less than 19 mm. The loads that should be considered in computing the gross soil pressure on the rafts are the dead load of the structure including the raft, and the maximum live load that is likely to be active. The surcharge due to the weight of the soil between the surround ground surface and the base level of the foundation should be subtracted from the gross pressure to obtain the net pressure for comparison with the allowable soil pressure. In light of the structural loads and the excavation depth, the net soil pressure at the base assumed to be approximately 135 kPa for the 8-storey with one level of underground parking. Hence, the maximum total settlement of a rigid raft supporting the mixed-used buildings would be approximately 35 mm.

Given the conditions encountered in the boreholes BH1, BH2, and BH6, the use of raft foundations should provide a practical approach for the 8-storey retirement residence building with two levels of underground parking (Phase 3). Because of the relatively large size of the rafts, the bearing capacity failure at ULS condition is too remote to require consideration. The allowable soil pressure for design at SLS condition is estimated to be 210 kPa using the average SPT-N value within the inference zone. A modulus of subgrade reaction of 4.2 MN/m^3 could be used for structural design of the raft foundations. The proposed values for the soil resistance and modulus of subgrade reaction are based on the assumption that the rafts will to be founded on the natural soil at 5.8 to 6.0 mbgl. The values given for SLS geotechnical resistances are based on maximum settlement values of 50 mm. Total differential settlements within a building should also be less than 19 mm. The loads that should be considered in computing the gross soil pressure on the rafts are the dead load of the structure including the raft, and the maximum live load that is likely to be active. The surcharge due to the weight of the soil between the surround ground surface and the base level of the foundation is subtracted from the gross pressure to obtain the net pressure for comparison with the allowable soil pressure. In light of the structural loads and the excavation depth, the net soil pressure at the base assumed to be approximately 125 kPa for the 8-storey building with two level of underground parking. Hence, the maximum total settlement of a rigid raft supporting the retirement residence buildings would be approximately 30 mm.

Prior to pouring concrete for the footings, the footing subgrade 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. 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.

Soil resistance beneath the footprint of the structures may be found to differ, during construction, from those at the borehole locations. Given this scenario, the contractor should contact the design engineer for recommendations as how to best proceed and what changes if any, should be made.

The exposed subgrade 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 A or equivalent materials compacted to 100% Standard Proctor Maximum Dry Density (SPMDD). 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. Full-time supervision and in-situ density testing should be carried out by a geotechnical engineer during placement of engineered fill beneath all structures and settlement sensitive areas.

Backfilling of foundations shall be carried out with approved OPSS Granular B material provided. It can be placed in maximum 300 mm loose lifts and compacted to a minimum of 98% SPMDD. Filling should continue until the design subgrade elevations are obtained.

7.3 Slab-On-Grade Floor Using Engineered Fill

Prior to construction of the floor slab, all topsoil, construction debris and deleterious materials must be removed from the ground surface. To create a stable working surface and to distribute loadings, compacted OPSS Granular A or equivalent should be placed below all floor slabs. The compacted OPSS Granular A or equivalent should be 200 mm thick at minimum, compacted to 100% SPMDD.

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 module of vertical subgrade reaction (K_s) for granular material over the encountered subgrade materials is approximated to be $20\ MN/m^3$. This value should be modified by appropriate shape and depth factors to determine the vertical sub grade modulus (K_s) for slabs and bases.

Soil resistance beneath the footprint of the structures may be found to differ, during construction, from those at the borehole locations. Given this scenario, the contractor should contact the design engineer for recommendations as how to best proceed and what changes if any, should be made.

7.4 Earthquake Design Parameters

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 Ontario Building Code (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 SPT-N values from borehole information, the subsurface stratigraphy generally comprises of stiff soil. On this basis, the site designation for seismic analysis is **Class D** according to Table 4.1.8.4.A from the quoted code.

7.5 Lateral Earth Pressure on Walls

All below grade walls and retaining walls should be designed to withstand lateral earth pressures. The Lateral earth pressures may be calculated using the following equation:

$$p = k(\gamma h + q)$$

Where p is lateral earth pressure, k is coefficient of lateral earth pressure, γ is backfill total unit weight, h is depth from the ground surface and q is surcharge at ground surface adjacent to the wall. Recommended design values are presented in Table 8 below. It is expected that all below grade wall would be rigid, as such, the at-rest coefficient of earth pressure, k_0, is recommended in the calculation of the lateral earth pressures. Where some movement can be accommodated for retaining walls, the active earth pressure coefficient, k_a, can be used.

The above expression assumes that backfill consisting of free-draining granular material with a drainage system to prevent the build-up of hydrostatic pressure behind the wall. If this is not possible, then combined hydrostatic and lateral earth pressures should be applied using water unit weight of 9.8 kN/m^3.

Table 8 – Lateral Earth Pressure Parameters

Backfill Type	Lateral Earth Pressure Coefficient			Total Unit Weight, kN/m^3
	Active, K_a	At-Rest, K_0	Passive, K_p	
Granular Material	0.33	0.50	3.0	20
Lean Clay	0.53	0.69	1.9	18

Backfill behind retaining walls should consist of non-frost susceptible, free-draining granular materials in accordance with OPSD 3101.150. The granular backfill should be compacted to at least 98% SPMDD, placed in maximum 200 mm lifts. The backfill should be brought up around the exterior of the walls as evenly as possible to prevent differential pressures.

7.6 Temporary Shoring System

The excavation for the proposed development will extend about 3.0-3.5 mbgl, therefore, vertical or near vertical excavation walls may be needed. The contractor is fully responsible for the detailed design and performance of the temporary shoring systems. This report only provides some general guidelines on possible options for the shoring to be utilized by the designers for evaluating the impacts of the shoring design and site works as well as to assess the potential for impacts of this shoring on the adjacent properties.

The shoring method(s) utilized to support the excavation sides must take into account the soil and stratigraphy, the permissible displacement of the shoring, the groundwater conditions, and the method of construction. In addition, the potential ground movements associated with the excavation and construction of the shoring system as well as their impact on adjacent structures and utilities must be incorporated in the shoring design.

For preliminary design purposes, a soldier pile and lagging system can be deemed as a suitable shoring method that may be considered for the proposed 3.0-3.5 metres deep excavation at the subject site. The shoring system shall be provided with appropriate lateral support. For the soldier pile and lagging, excavation for placement of the lagging shall be performed in such a manner that the lagging is tight against the excavation face. Any voids behind the lagging shall be backfilled with suitable compacted material.

The temporary structure’s vertical support members and horizontal restraints for the excavation should be designed in accordance with the Canadian Foundation Engineering Manual 4th edition (CFEM). The design and installation procedure should be reviewed by a professional engineer prior to the commencement of construction. The installation operation must be monitored on a full-time basis by geotechnical personnel to ensure allowable bearing pressure, foundation elevations and alignment as per National Building Code (NBC) requirements. The pile/caisson base should be inspected prior to installation of steel pile or placement of concrete. The shoring system should be continuously monitored for movement after installation, to assure that displacements remain within the specified limits.

7.7 Groundwater Control

For foundation excavations extending below the groundwater level, it will be necessary to lower and maintain the groundwater level below the excavation base. As described in the section 6.5, Water levels were encountered at a depth of approximately 2.7 to 4.3 mbgl. Therefore, the excavation area and foundation zone for the underground parking will be most likely in a wet area, hence, suitable dewatering technique such as sump pump or sewer with a check valve should be employed to make the construction area dry. The groundwater table is subject to seasonal fluctuations and may be at a higher level during wet weather periods. 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)

γ_w = unit weight of water (9.8 kN/m^3)

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.

7.8 Basement Drainage

Water levels were encountered at a depth of approximately 2.7 to 4.3 mbgl. Therefore, varying amounts of groundwater seepage may be encountered in the excavation in some areas (depending upon the depth of excavation). To assist in maintaining dry conditions in the underground parking garage from seepage, it is recommended that exterior grades around the excavated area be sloped away at a 2 percent gradient or more, for a distance of at least 1.2 m. As well, perimeter foundation drains should be provided, consisting of perforated pipe with filter fabric (minimum 100 mm diameter) surrounded by a granular filter (minimum 150 mm thick), and freely out-letting. The granular filter should consist of HL8 Coarse Aggregate or OPSS 1004 19 mm Clear Stone surrounded by a filter fabric (Terrafix 270R or equivalent).

The underground parking garage walls must be provided with damp-proofing provisions in conformance to Section 9.13.2 of the Ontario Building Code. The underground parking wall backfill for a minimum lateral distance of 0.6 m out from the wall should consist of free-draining granular material (OPSS 1010 Granular B), or provided with a suitable alternative drainage cellular media. The perimeter drain installation and outlet provisions must conform to the plumbing code requirements.

The size of the sump should be adequate to accommodate the water seepage and stormwater inflow. The sub-floor drainage system should be designed to prevent the possibility of back-flow. A duplex pumping arrangement (main pump with a provision of a backup pump) on emergency backup power is recommended. The pumps should have sufficient capacity to accommodate a maximum peak flow of water. It is common to provide a storm sump pump with a nominal 200 liters per minute pumping capacity using an industrial pump to remove water from the system, when required. This flow is not anticipated to be a sustained flow, but could be achieved under certain peak flow conditions.

7.9 Site Grading and Engineered Fill Construction

Site grading operations involving "cut and fill" procedures in the order of $\pm 2\ \mathrm{m}$ are expected through the site. It is recommended to construct engineered fill in areas to be raised in order to suitably support the future fire route, infrastructure servicing and lightly loaded building structures.

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.

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 imported 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 a wet spring or fall.

7.10 Site Servicing

7.10.1 Excavation Conditions

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. 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 may be sloped not steeper than one vertical to one horizontal throughout. The excavation side slopes should be suitably protected from erosion processes. For the conventional excavation depth, it is anticipated to encounter major water flow into the excavation. Should unstable and/or wet conditions be encountered, side slopes are to be flattened to a stable configuration. Note that Type 3 soil exerts substantial fluid pressure on its supporting system. The geotechnical engineer should be retained to examine and inspect cut slopes to ensure construction safety.

7.10.2 Pipe Bedding

The native and re-compacted fill soil will generally provide suitable subgrade support to sewer and watermain servicing provided that the integrity of the base of the trench excavations can be maintained during construction. Any unsuitable soils exposed at the pipe subgrade should be sub-excavated and replaced with a minimum 150 mm bedding thickness of OPSS Granular A, compacted to at least 98% SPMDD. The bedding requirements for the services should be in accordance with Ontario Provincial Standard Drawings (OPSD) standards and the local County's Standards. Granular "A" should be used to backfill around the pipe to at least 150 mm above the top of the pipe. From the springline to 300 mm above the obvert of the pipe, sand cover shall be used. Particular attention should be given to ensure material placed beneath the haunches of the pipe is adequately compacted.

7.10.3 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 at or near the optimum water content for compaction 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.

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 300 mm lifts and compacted to a minimum of 98% SPMDD. 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. It is expected that most material will have to be imported. 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 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.

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. 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.

7.10.4 Pavement Structures

It is our understanding from the proposed development that a new parking lot will be constructed for this project. The recommended pavement structure is outlined in Table 9, based on the anticipated traffic volume and subgrade conditions. The recommended pavement structure should be considered for preliminary design purposes only.

It is assumed that pavement construction will be carried out under dry periods and the subgrade will be stable under the load of construction equipment. If the subgrade is unstable or wet, additional thickness of sub- base course material may be required. It should be noted that the recommended pavement structure is not intended to support heavy construction vehicles such as concrete trucks. Consequently, heavy construction traffic should be limited to areas with suitable temporary access roads. The access roads shall consist of a minimum of 450 mm of stony Granular B material placed on a woven geogrid to preclude mixing of the subgrade into the Granular B. A surface coat of recycled asphalt shall be placed on the surface to provide a seal.

Table 9 – Minimum Pavement Structure Requirements

Pavement layer	Thickness (mm)	Material
Surface Course Asphalt	50	OPSS H.L3
Binder Course Asphalt	60	OPSS H.L8
Base Layer	200	OPSS Granular A
Subbase Layer	350	OPSS Granular B

The granular base and sub-base layers should be uniformly compacted to 100% SPMDD. The asphalt materials should be compacted to a minimum of 92% of the Marshal Maximum Relative

Density (MRD), as tested by using nuclear density gauge.

Prior to placing the pavement sub-base layer, the subgrade should be prepared and heavily proof-rolled under the supervision of the geotechnical engineer. Any weak or soft areas encountered at the original surface must be further sub-excavated and replaced with suitable approved backfill compacted to 98% SPMDD to provide uniform subgrade support condition. The subgrade should be compacted to 98% SPMDD for at least the upper 500 mm. Stringent compaction and placement control procedures shall be maintained to ensure uniform subgrade moisture and density conditions are achieved.

It should be noted that even with well-compacted trench backfill, some settlement can be expected after construction. In this regard, surface course asphalt shall be placed at least one year after trench backfill is completed.

The finished pavement surface should be graded to promote runoff to designated surface drainage areas and catch basins. Subdrains should be installed to intercept excess subsurface moisture and prevent subgrade softening. To minimize problems of differential movement between the pavement and catch basins/manholes due to frost action, the backfill around the structures should consist of free draining granular. It is recommended to install longitudinal subdrain with positive drainage outlets at the subgrade level along the edges of the roadway construction. The subdrain stubs should be extended at least 10 m from catch basins, along the uphill sides.

7.10.5 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 100 mm thick layer of compacted (minimum 98% SPMDD) Granular A is recommended below sidewalk slabs.

8.0 LIMITATIONS OF REPORT

This report has been prepared for Columbia Square Inc. (the Client), who retained the services of A&A to conduct a geotechnical investigation for a proposed development on a property located at 14245 Highway 50, 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 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.

LICENSED PROFESSIONAL ENGINEER
M.M. HEIDARI SARVESTANI
100521871
PROVINCE OF ONTARIO

December 16, 2021
Mehdi Heidari, Ph.D., 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 14245 Highway 50, Caledon, ON

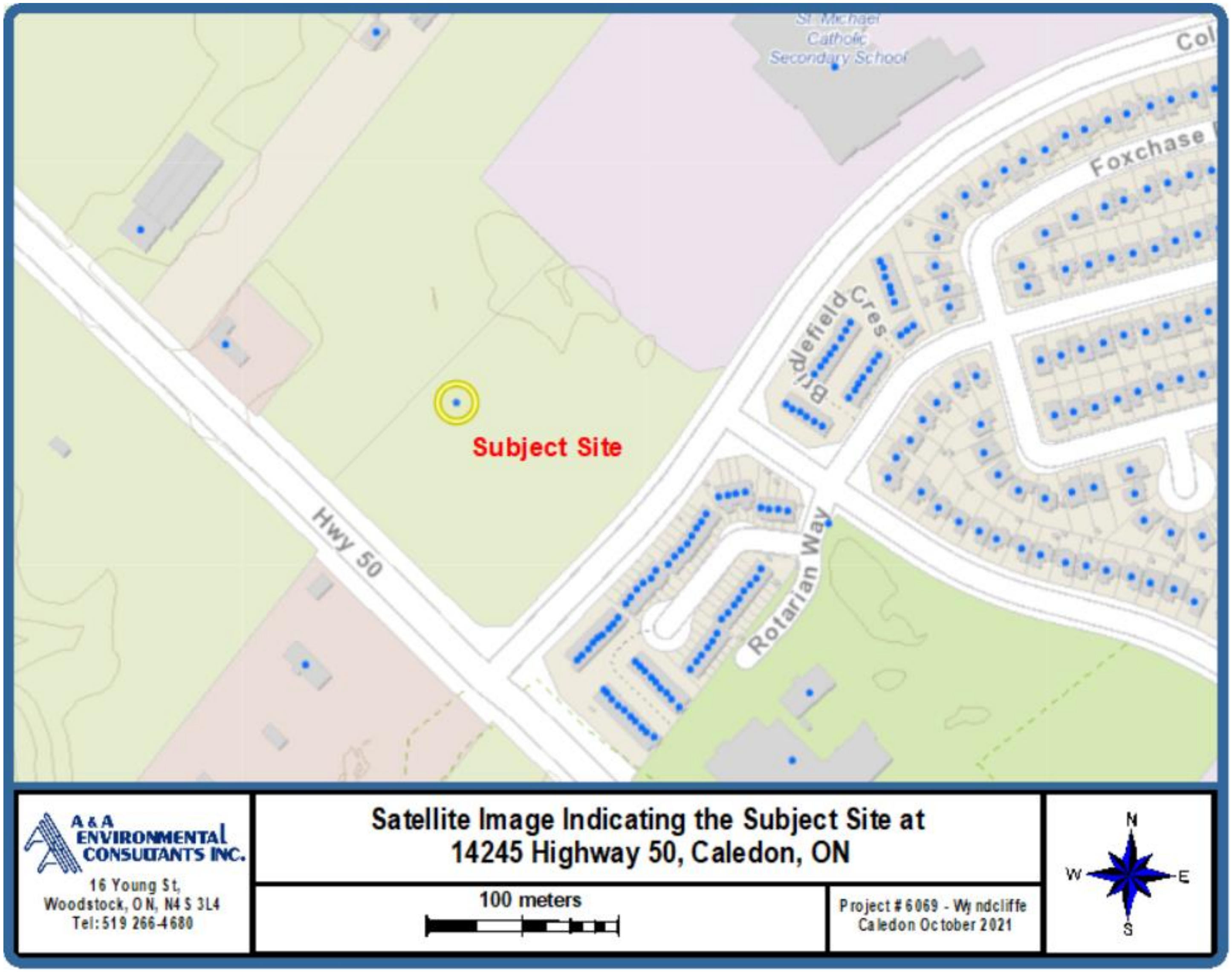


Figure 2 – Topographic Map of Subject Study Area

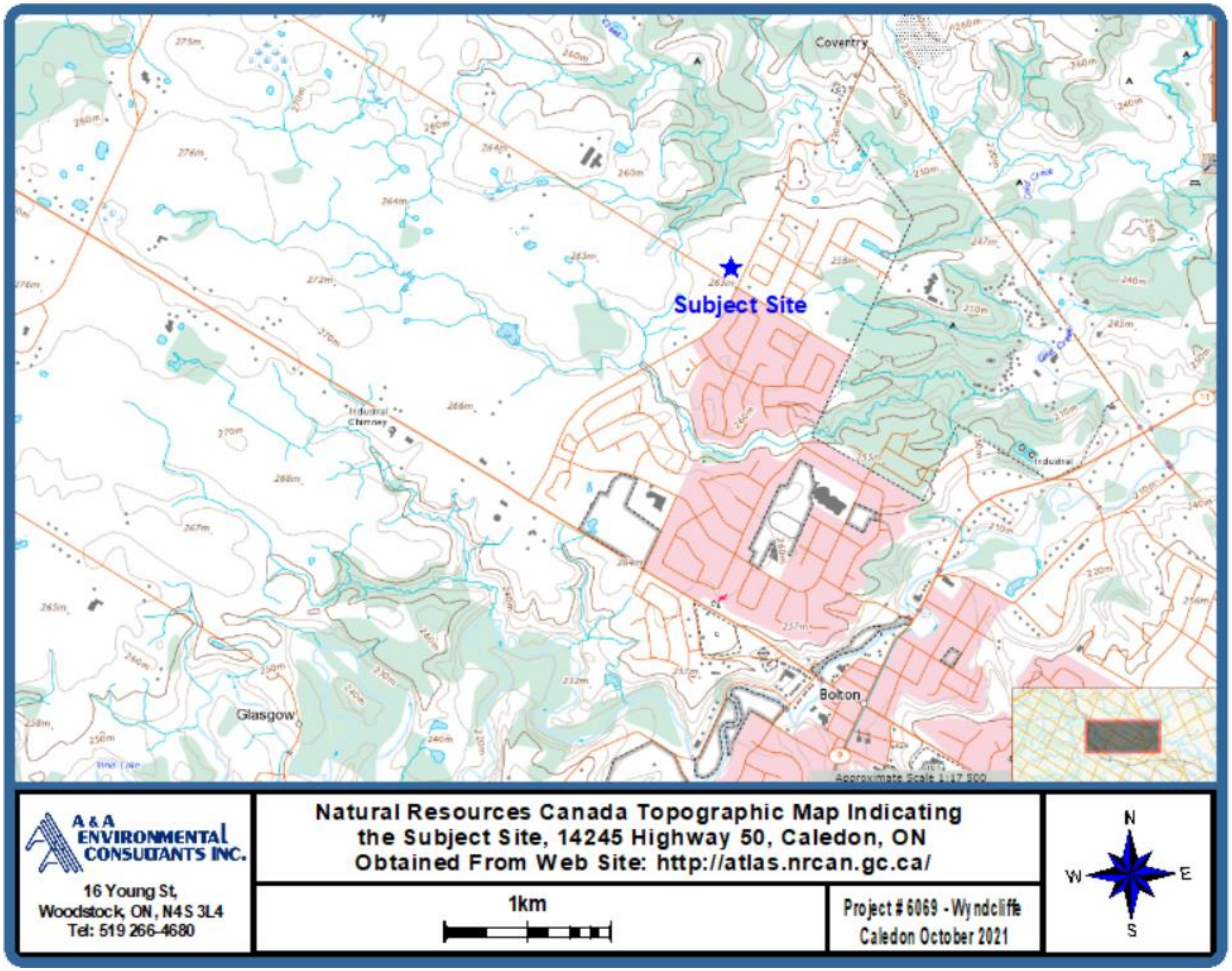
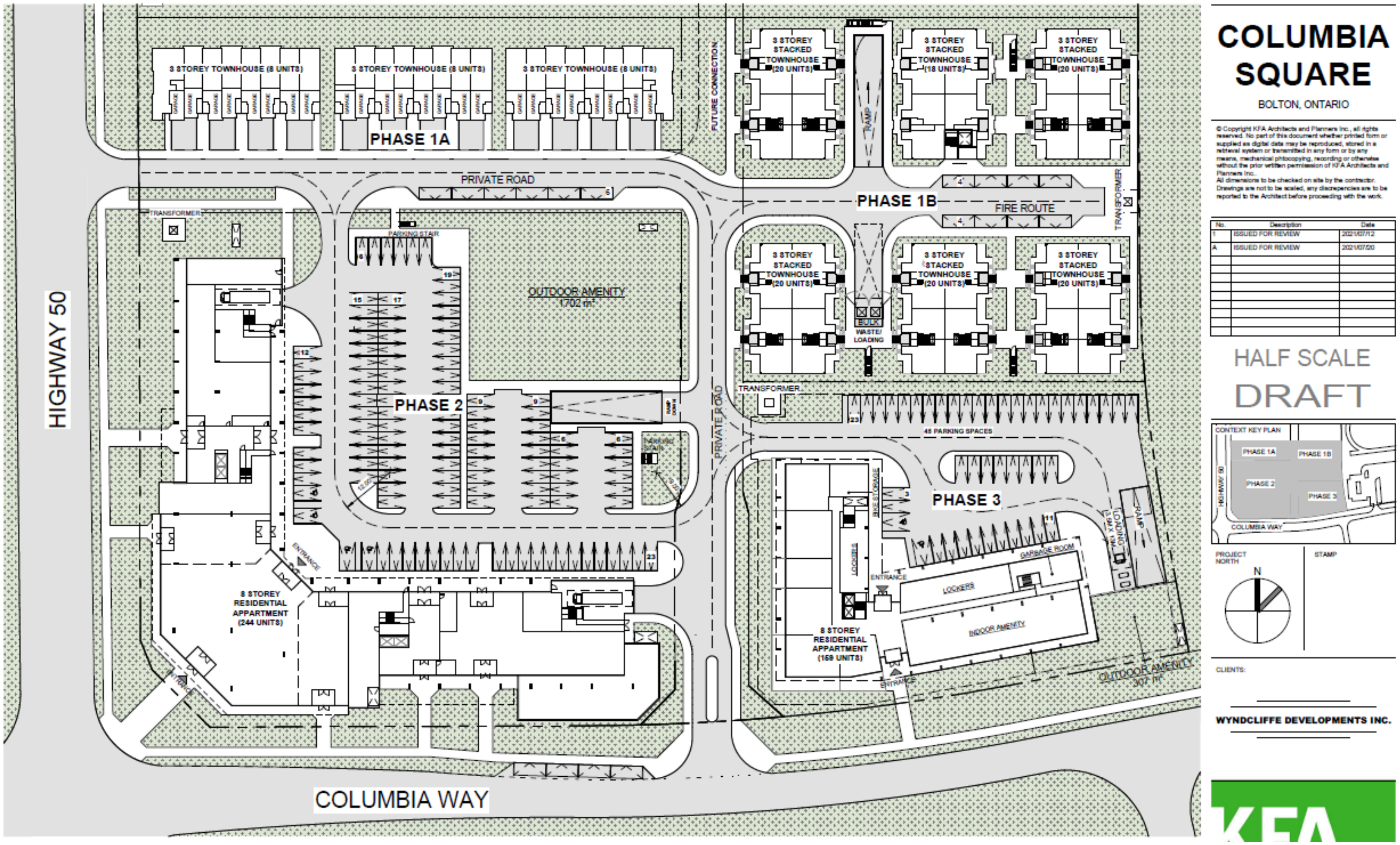


Figure 3 – Proposed Development



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All dimensions to be checked on site by the contractor. Drawings are not to be scaled, any discrepancies are to be reported to the Architect before proceeding with the work.

No.	Description	Date
1	ISSUED FOR REVIEW	2021/07/12
A	ISSUED FOR REVIEW	2021/07/20

Figure 4 – Geotechnical Borehole Locations, Satellite Image

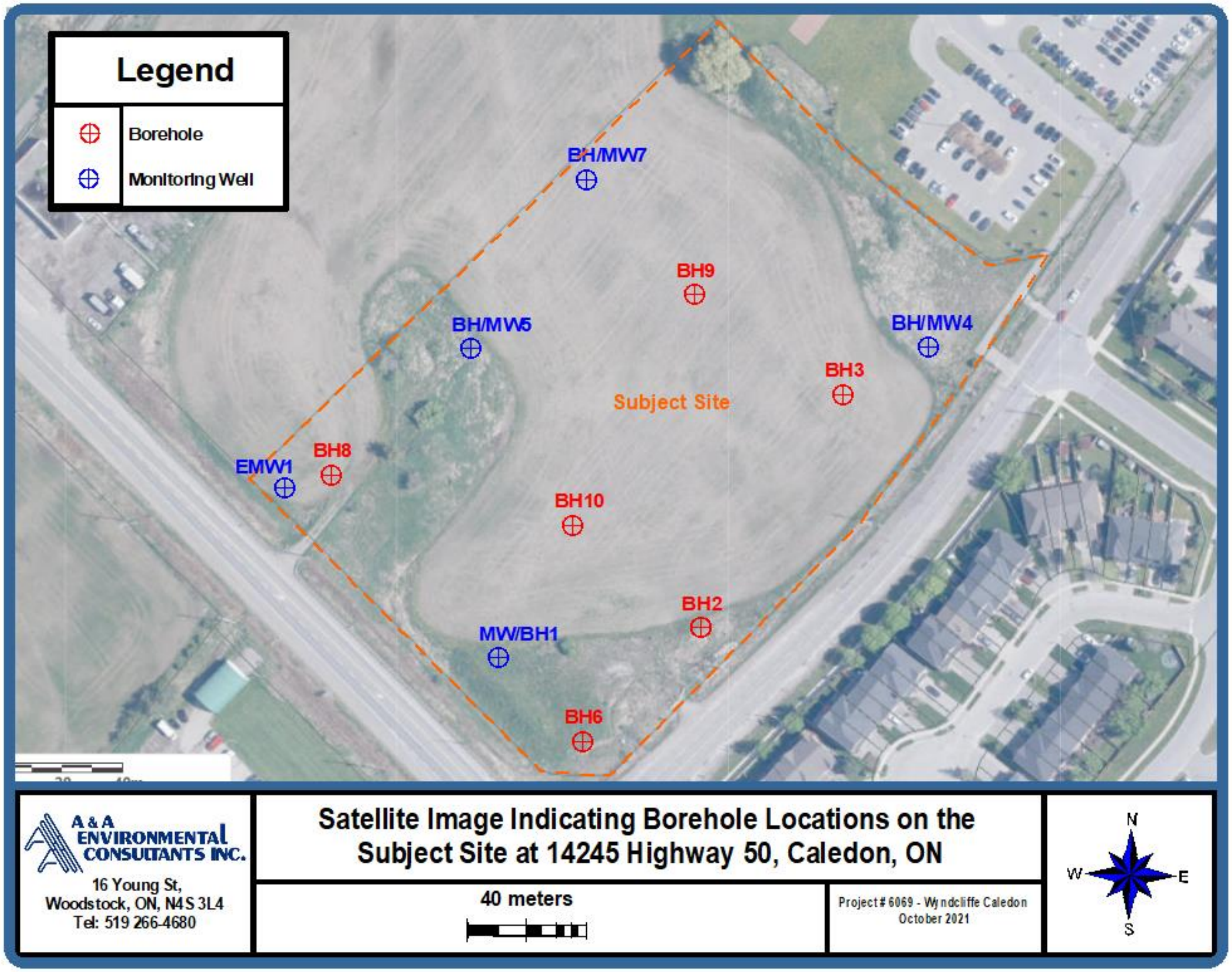
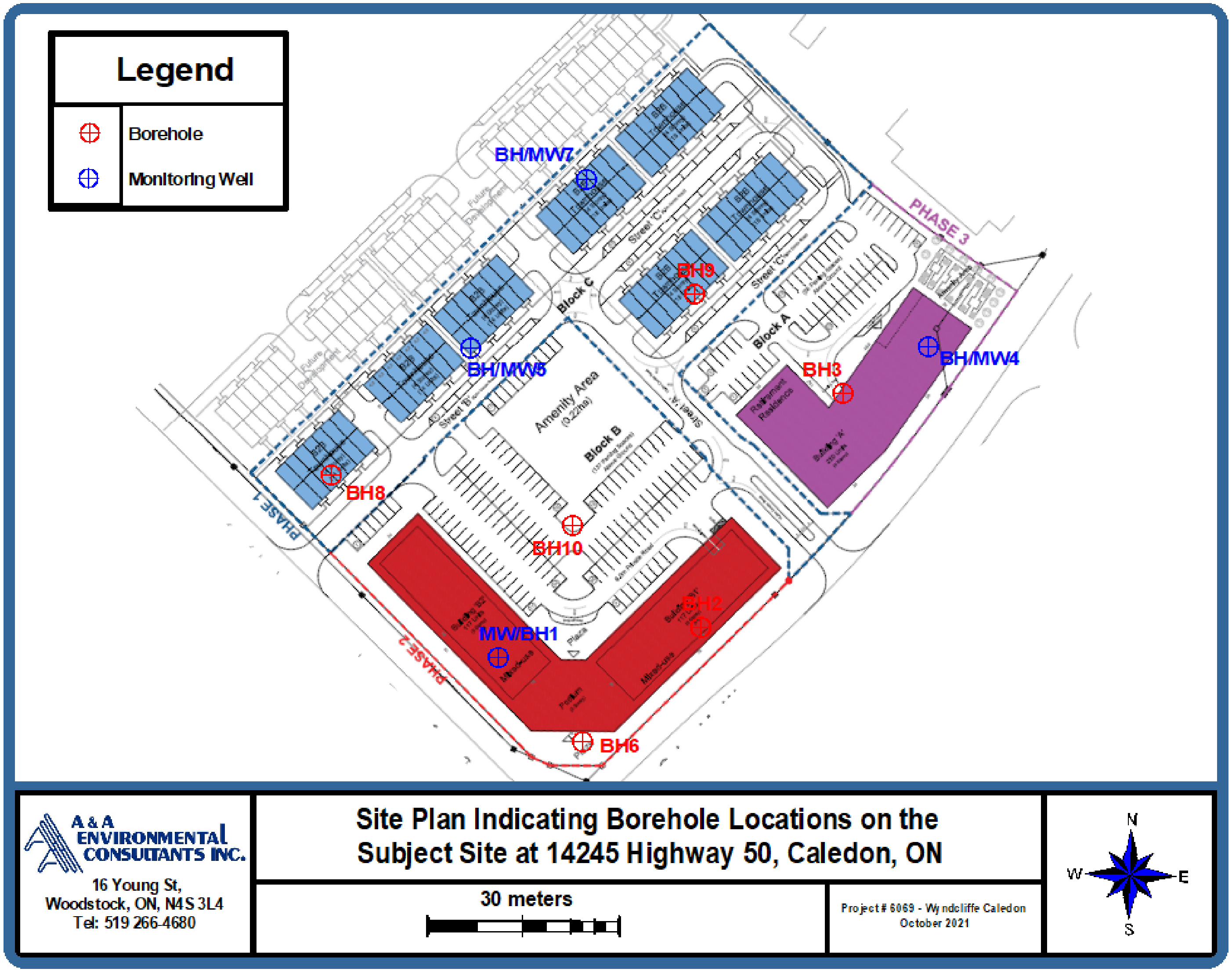


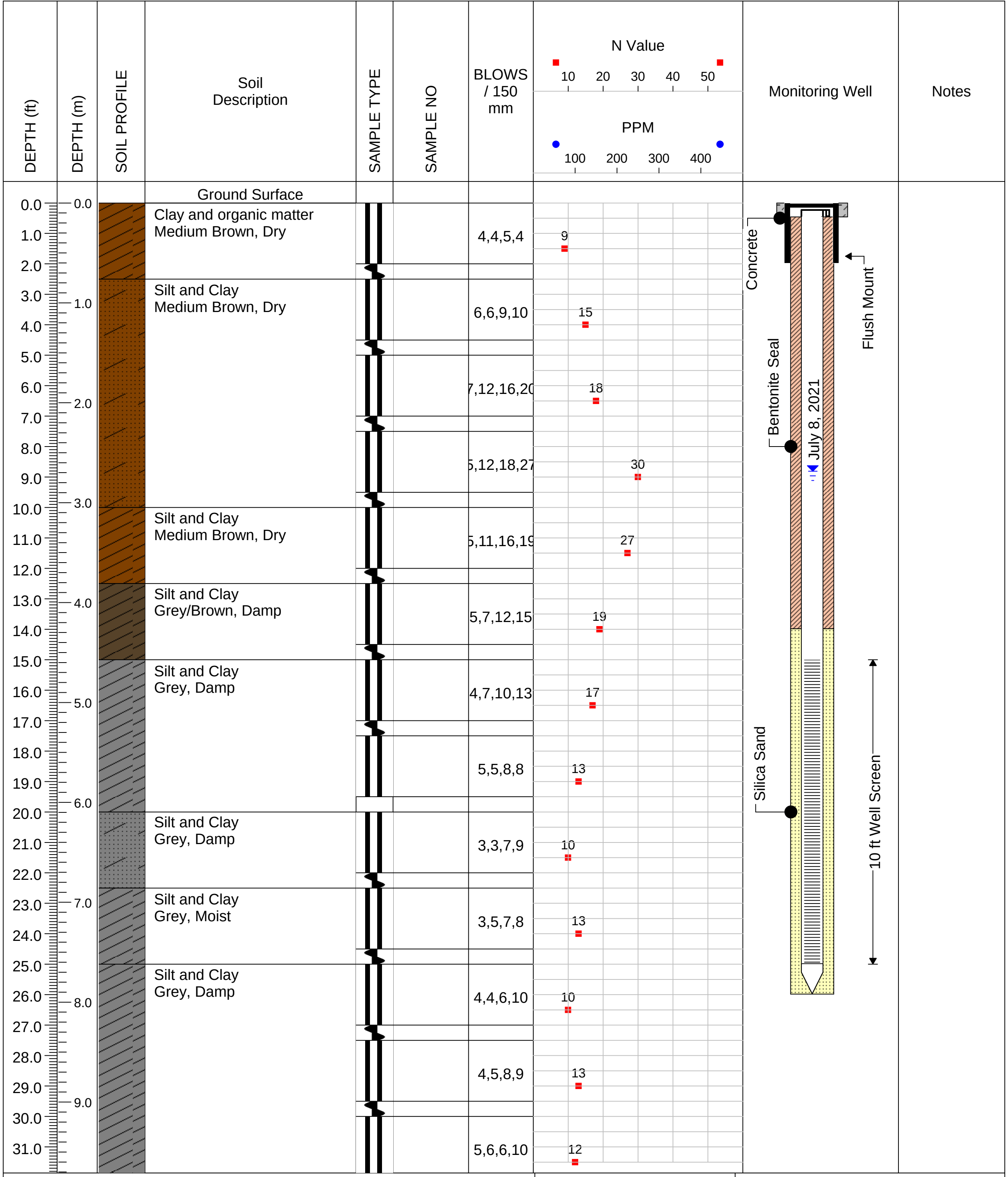
Figure 5 – Geotechnical Borehole Locations, Site Plan



APPENDIX B – Borehole Logs and Explanation of Terms and Symbols

PROJECT: Wyndcliffe Caledon	**BH LOCATION:** Central West Boundary	**BOREHOLE NO: BH/MW-1**
PROJECT NO: 6069	**LOCATION:** 14245 Highway 50, 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



Soil Description	BLOWS / 150 mm	N Value
Clay and organic matter Medium Brown, Dry	4,4,5,4	9
Silt and Clay Medium Brown, Dry	6,6,9,10	15
	7,12,16,20	18
	5,12,18,27	30
Silt and Clay Medium Brown, Dry	5,11,16,19	27
Silt and Clay Grey/Brown, Damp	5,7,12,15	19
Silt and Clay Grey, Damp	4,7,10,13	17
	5,5,8,8	13
Silt and Clay Grey, Damp	3,3,7,9	10
Silt and Clay Grey, Moist	3,5,7,8	13
Silt and Clay Grey, Damp	4,4,6,10	10
	4,5,8,9	13
	5,6,6,10	12

A & A ENVIRONMENTAL CONSULTANTS INC.
16 Young Street Woodstock, ON

LOGGED BY: J. Osborne	**COMPLETION DEPTH:** 60 Feet
REVIEWED BY: A. Rasoul	**DRILL METHOD:** SPT
DRILL DATE: May 25, 2021	**PAGE:** 1 of 2

PROJECT: Wyndcliffe Caledon	**BH LOCATION:** Central West Boundary	**BOREHOLE NO: BH/MW-1**
PROJECT NO: 6069	**LOCATION:** 14245 Highway 50, 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): 32.0, 33.0, 34.0, 35.0, 36.0, 37.0, 38.0, 39.0, 40.0, 41.0, 42.0, 43.0, 44.0, 45.0, 46.0, 47.0, 48.0, 49.0, 50.0, 51.0, 52.0, 53.0, 54.0, 55.0, 56.0, 57.0, 58.0, 59.0, 60.0, 61.0, 62.0, 63.0

DEPTH (m): 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0, 17.0, 18.0, 19.0

N Value: 10, 20, 30, 40, 50

PPM: 100, 200, 300, 400

Soil Description	SAMPLE TYPE	SAMPLE NO	BLOWS / 150 mm	N Value	Monitoring Well	Notes
				12		
Silt and Clay Grey, Moist			3,5,8,9	13		
			2,4,5,6	9		
			5,5,6,9	11		
			2,3,6,7	9		
			3,3,5,4	8		
			0,2,4,3	6		
Clayey Silt Grey, Damp			2,7,11,13	18		
Clayey Silt Grey, Wet			5,15,16,18	31		
Clayey Silt Grey, Moist Clayey Silt Grey, Wet			6,10,14,21	24		
Clayey Silt Grey, Moist			4,13,15,20	28		
Clayey Silt Grey, Dry			3,13,19,28	32		
End of Log						

Notes
Borehole completed at 60 Feet
Monitoring well installed at 20.5

A & A ENVIRONMENTAL CONSULTANTS INC.
16 Young Street Woodstock, ON

LOGGED BY: J. Osborne	**COMPLETION DEPTH:** 60 Feet
REVIEWED BY: A. Rasoul	**DRILL METHOD:** SPT
DRILL DATE: May 25, 2021	**PAGE:** 1 of 2

PROJECT: Wyndcliffe Caledon	**BH LOCATION:** West Portion of Southwest Boundary	**BOREHOLE NO: BH-2**
PROJECT NO: 6069	**LOCATION:** 14245 Highway 50, 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	N Value (10 20 30 40 50) / PPM (100 200 300 400)	Monitoring Well	Notes
			Ground Surface						
0.0	0.0		Sand, Clay, Rock, Organic Matter Light Brown, Dry			5,8,4,5	6		
			Clay, some organic matter Light Brown, Dry			3,7,9,11	5		
			Clay, trace sand Light Brown, Dry			6,8,13,14	8		
			Silt and Clay Grey/Brown, Dry			5,12,14,16	9		
			Silt and Clay Grey, Damp			2,4,5,5	9		
			Silt and Clay Grey, Damp			2,4,6,7	11		
			Silt and Clay Grey, Damp			3,7,5,5,	8		
			Silt and Clay Grey, Moist			2,4,6,6	10		
			Silt and Clay Grey, Moist			1,2,4,7	12		
						1,2,3,5	16		
						2,4,4,5	21		
						1,4,5,8	26		

DEPTH (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, 16.0, 17.0, 18.0, 19.0, 20.0, 21.0, 22.0, 23.0, 24.0, 25.0, 26.0, 27.0, 28.0, 29.0, 30.0

DEPTH (m): 0.0, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0

A & A ENVIRONMENTAL CONSULTANTS INC.
16 Young Street Woodstock, ON

LOGGED BY: J. Osborne	**COMPLETION DEPTH:** 60 Feet
REVIEWED BY: A. Rasoul	**DRILL METHOD:** SPT
DRILL DATE: May 25, 2021	**PAGE:** 1 of 2

PROJECT: Wyndcliffe Caledon	**BH LOCATION:** West Portion of Southwest Boundary	**BOREHOLE NO: BH-2**
PROJECT NO: 6069	**LOCATION:** 14245 Highway 50, 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): 31.0, 32.0, 33.0, 34.0, 35.0, 36.0, 37.0, 38.0, 39.0, 40.0, 41.0, 42.0, 43.0, 44.0, 45.0, 46.0, 47.0, 48.0, 49.0, 50.0, 51.0, 52.0, 53.0, 54.0, 55.0, 56.0, 57.0, 58.0, 59.0, 60.0, 61.0

DEPTH (m): 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0, 17.0, 18.0

N Value: 10, 20, 30, 40, 50

PPM: 100, 200, 300, 400

Soil Description	SAMPLE NO	BLOWS / 150 mm	N Value	Monitoring Well	Notes
Silt and Clay Grey, Damp		1,4,5,6	9		
		3,5,6,7	10		
		2,4,4,6	13		
		3,4,6,7	10		
Silt and Clay Grey, Damp		2,6,5,5	11		
Clayey Silt Grey, Damp		5,8,9,11	17		
		5,8,12,14	20		
		4,8,10,17	18		
Clayey Silt Grey, Damp		3,11,12,14	23		
		5,7,14,21	21		
		3,16,21,18	37		
		7,10,11,19	21		
End of Log					

Notes
Borehole completed at 60 Feet

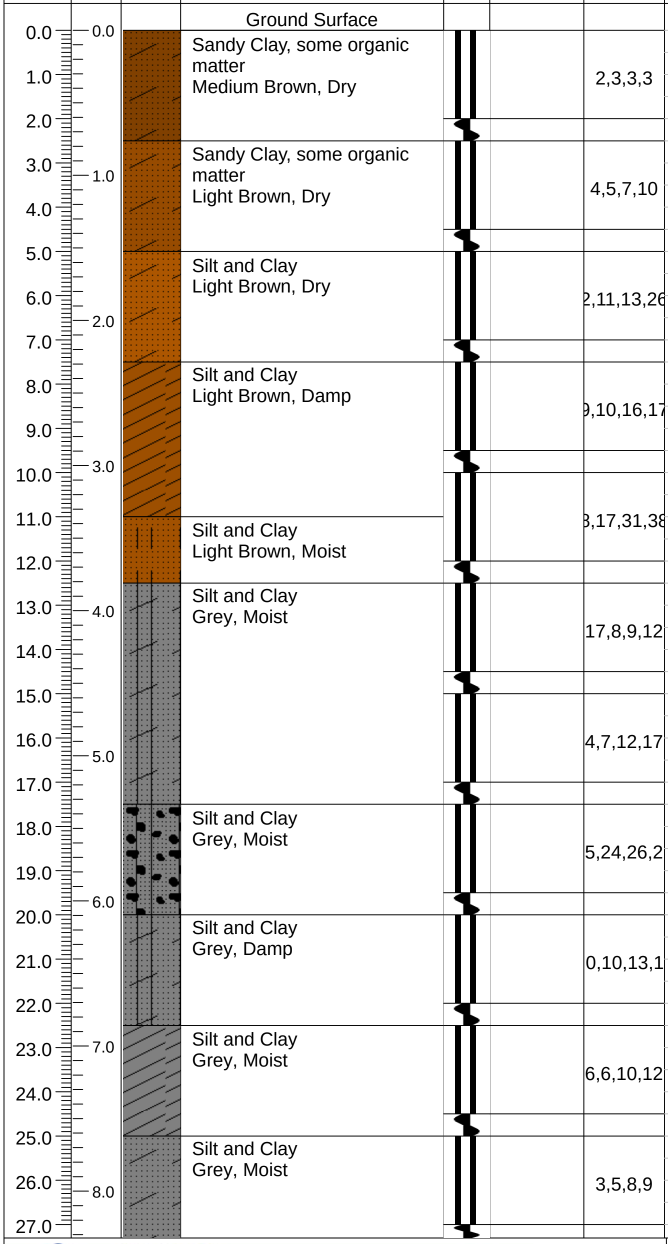
A & A ENVIRONMENTAL CONSULTANTS INC.
16 Young Street Woodstock, ON

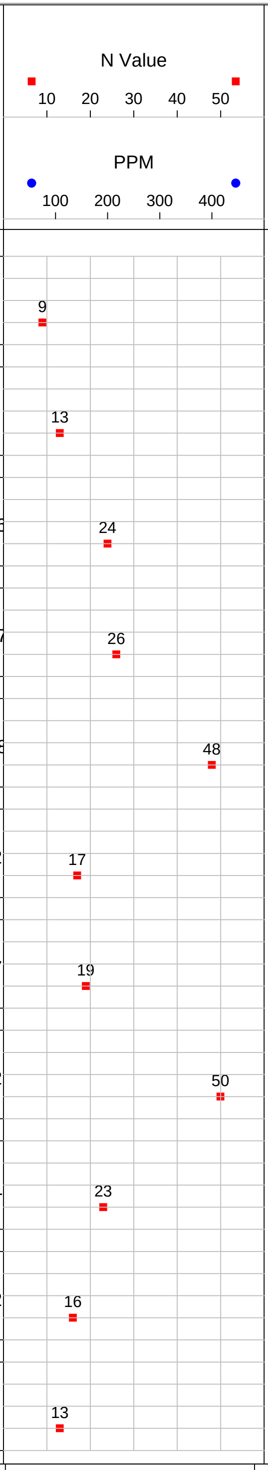
LOGGED BY: J. Osborne	**COMPLETION DEPTH:** 60 Feet
REVIEWED BY: A. Rasoul	**DRILL METHOD:** SPT
DRILL DATE: May 25, 2021	**PAGE:** 1 of 2

PROJECT: Wyndcliffe Caledon	**BH LOCATION:** Central East Quadrant	**BOREHOLE NO: BH-3**
PROJECT NO: 6069	**LOCATION:** 14245 Highway 50, 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

Soil Description	SAMPLE TYPE	SAMPLE NO	BLOWS / 150 mm	N Value	Monitoring Well	Notes
Ground Surface						
Sandy Clay, some organic matter Medium Brown, Dry			2,3,3,3	9		
Sandy Clay, some organic matter Light Brown, Dry			4,5,7,10	13		
Silt and Clay Light Brown, Dry			2,11,13,26	24		
Silt and Clay Light Brown, Damp			9,10,16,17	26		
			3,17,31,38	48		
Silt and Clay Light Brown, Moist						
Silt and Clay Grey, Moist			17,8,9,12	17		
			4,7,12,17	19		
Silt and Clay Grey, Moist			5,24,26,2	50		
Silt and Clay Grey, Damp			0,10,13,1	23		
Silt and Clay Grey, Moist			6,6,10,12	16		
Silt and Clay Grey, Moist			3,5,8,9	13		





A & A ENVIRONMENTAL CONSULTANTS INC.
16 Young Street Woodstock, ON

LOGGED BY: J. Osborne	**COMPLETION DEPTH:** 50 Feet
REVIEWED BY: A. Rasoul	**DRILL METHOD:** SPT
DRILL DATE: May 26, 2021	**PAGE:** 1 of 2

PROJECT: Wyndcliffe Caledon
BH LOCATION: Central East Quadrant
BOREHOLE NO: BH-3
PROJECT NO: 6069
LOCATION: 14245 Highway 50, Caledon ON
PROJECT MANAGER: T. Demers
COMPANY NAME: A&A Environmental Consultants Inc.

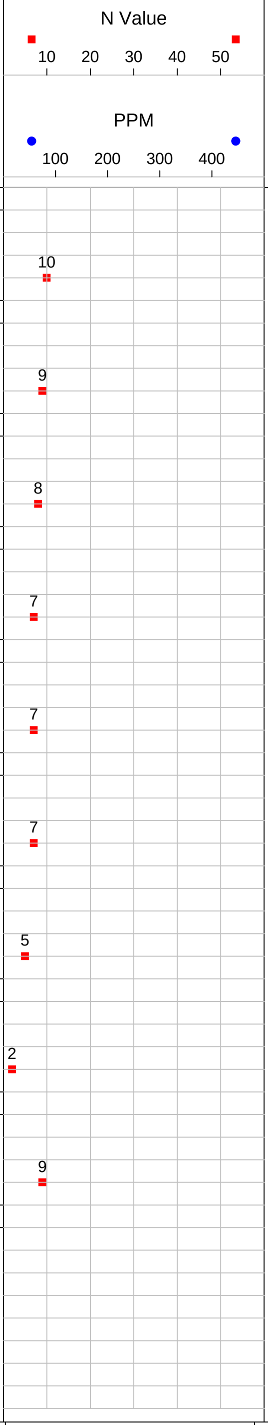
SAMPLE TYPE	SHELBY TUBE	CORE SAMPLE	SPT SAMPLE	GRAB SAMPLE	NO RECOVERY
BACKFILL TYPE	BENTONITE	PEA GRAVEL	SLOUGH	GROUT	DRILL CUTTINGS

DEPTH (ft): 28.0, 29.0, 30.0, 31.0, 32.0, 33.0, 34.0, 35.0, 36.0, 37.0, 38.0, 39.0, 40.0, 41.0, 42.0, 43.0, 44.0, 45.0, 46.0, 47.0, 48.0, 49.0, 50.0, 51.0, 52.0, 53.0, 54.0

DEPTH (m): 9.0, 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0

SOIL PROFILE	Soil Description
▨	
▨	Silt and Clay Grey, Moist
▨	Clayey Silt Grey, Wet
	End of Log

SAMPLE TYPE	SAMPLE NO	BLOWS / 150 mm	N Value
SPT SAMPLE		3,5,5,7	10
SPT SAMPLE		2,4,5,5	9
SPT SAMPLE		1,3,5,6	8
SPT SAMPLE		1,3,4,5	7
SPT SAMPLE		2,3,4,6	7
SPT SAMPLE		1,3,4,5	7
SPT SAMPLE		0,2,3,3	5
SPT SAMPLE		1,1,1,4	2
SPT SAMPLE		2,3,6,6	9



Monitoring Well

Notes

Notes
Borehole completed at 50 Feet

LOGGED BY: J. Osborne
COMPLETION DEPTH: 50 Feet
REVIEWED BY: A. Rasoul
DRILL METHOD: SPT
DRILL DATE: May 26, 2021
PAGE: 1 of 2

PROJECT: Wyndcliffe Caledon

BH LOCATION: Northwest of sidewalk

BOREHOLE NO: BH-4

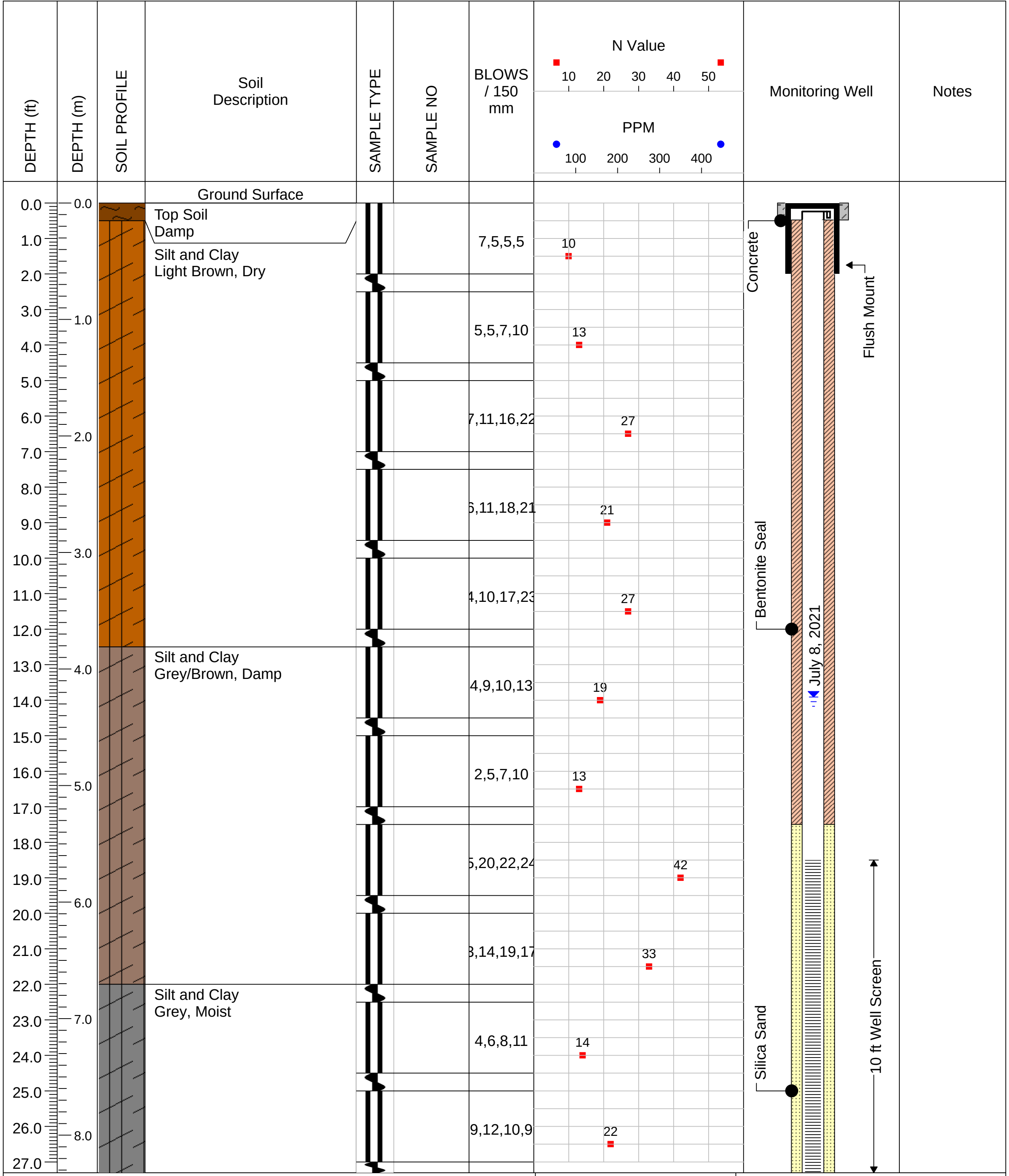
PROJECT NO: 6069

LOCATION: 14245 Highway 50, 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 CONSULTANTS INC.
16 Young Street Woodstock, ON

LOGGED BY: J. Osborne

REVIEWED BY: A. Rasoul

DRILL DATE: May 27, 2021

COMPLETION DEPTH: 50 Feet

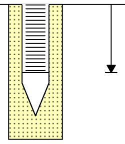
DRILL METHOD: SPT

PAGE: 1 of 2

PROJECT: Wyndcliffe Caledon	**BH LOCATION:** Northwest of sidewalk	**BOREHOLE NO: BH-4**
PROJECT NO: 6069	**LOCATION:** 14245 Highway 50, 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	N Value 10 20 30 40 50 / PPM 100 200 300 400	Monitoring Well	Notes
28.0									
29.0						3,4,8,10	12		
	9.0								
30.0									
31.0						3,4,7,9	11		
32.0									
	10.0		Silt and Clay Grey, Damp						
33.0									
34.0						3,4,6,7	10		
35.0									
36.0						2,3,6,7	9		
	11.0								
37.0									
38.0									
39.0						3,4,7,7	11		
	12.0								
40.0									
41.0						1,2,3,5	5		
42.0									
	13.0								
43.0									
44.0			Clayey Silt Grey, Damp			2,4,7,7	11		
45.0									
	14.0								
46.0						11,4,8,9	12		
47.0									
48.0									
49.0						4,7,10,11	17		
	15.0								
50.0			End of Log						
51.0									
52.0									
	16.0								
53.0									
54.0									



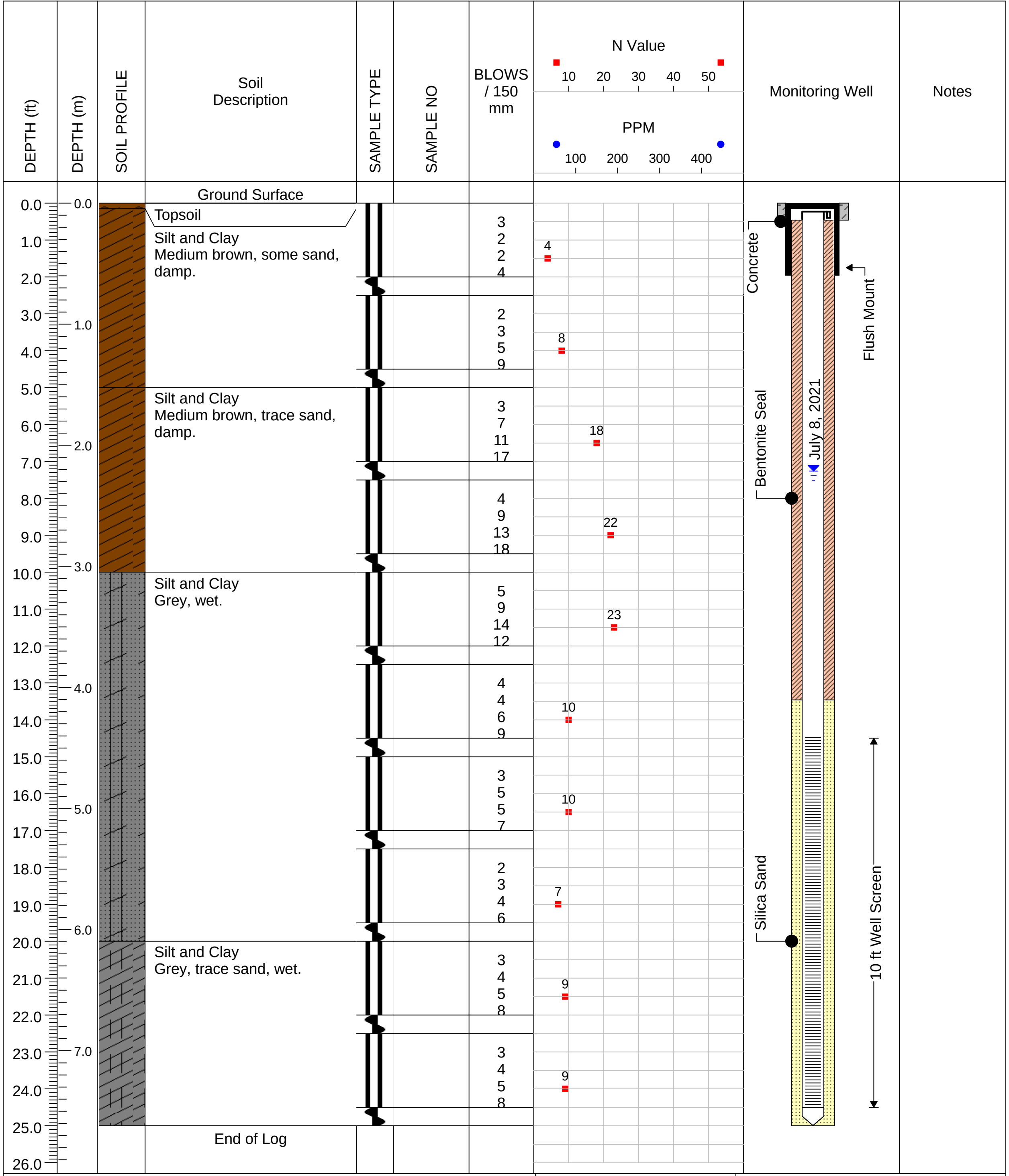
Notes
Borehole completed at 50 Feet
Monitoring well installed at 28.5 Feet

A & A ENVIRONMENTAL CONSULTANTS INC.
16 Young Street Woodstock, ON

LOGGED BY: J. Osborne	**COMPLETION DEPTH:** 50 Feet
REVIEWED BY: A. Rasoul	**DRILL METHOD:** SPT
DRILL DATE: May 27, 2021	**PAGE:** 1 of 2

PROJECT: Wyndcliffe Caledon
BH LOCATION: Near northwest boundary of site
BOREHOLE NO: BH/MW5
PROJECT NO: 6069
LOCATION: 14245 Highway 50, 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)	Soil Description
0.0	Ground Surface
0.0	Topsoil
0.5	Silt and Clay Medium brown, some sand, damp.
5.0	Silt and Clay Medium brown, trace sand, damp.
10.0	Silt and Clay Grey, wet.
20.0	Silt and Clay Grey, trace sand, wet.
25.0	End of Log

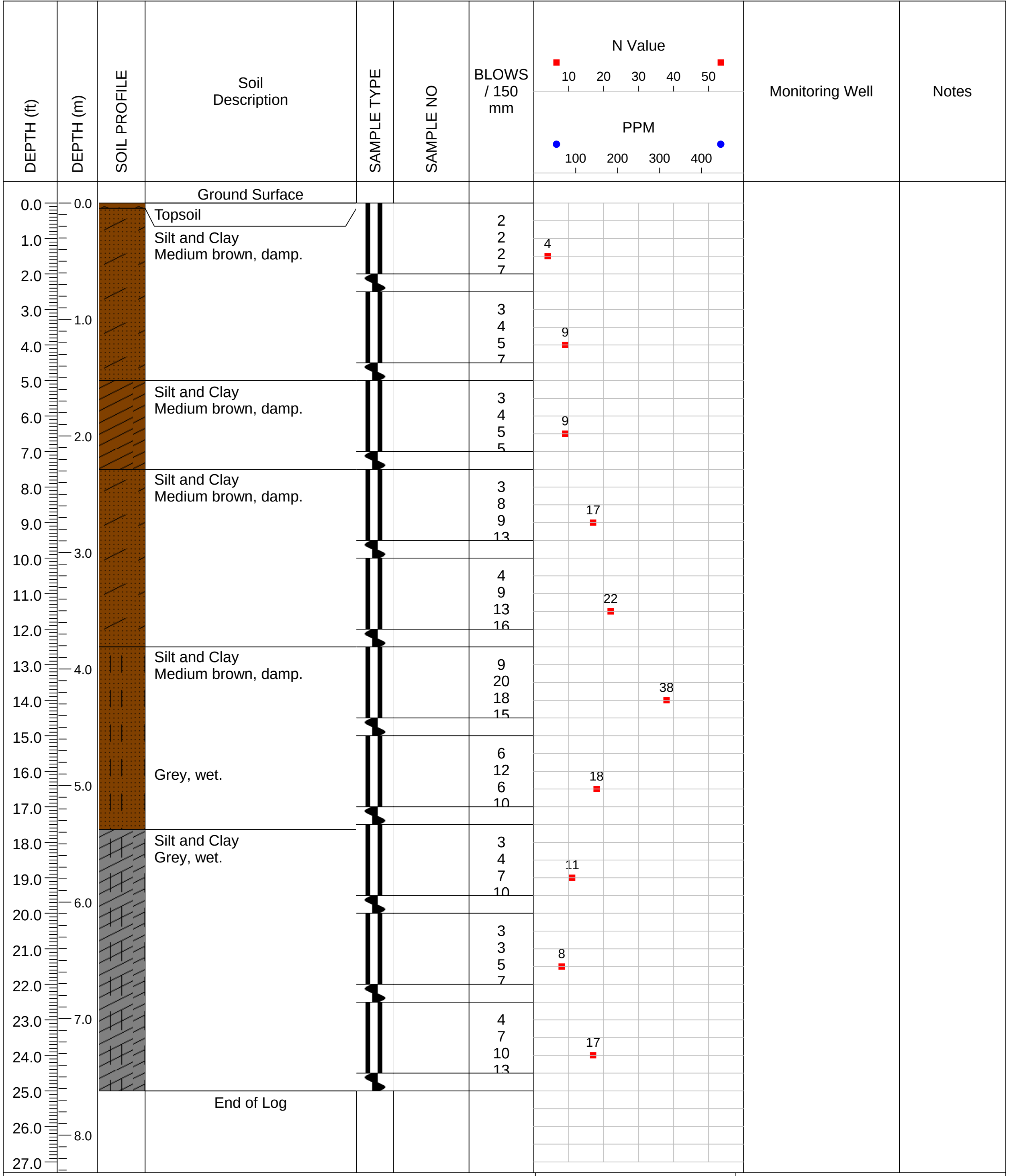
BLOWS / 150 mm	N Value
3 2 2 4	4
2 3 5 9	8
3 7 11 17	18
4 9 13 18	22
5 9 14 12	23
4 4 6 9	10
3 5 5 7	10
2 3 4 6	7
3 4 5 8	9
3 4 5 8	9

A & A ENVIRONMENTAL CONSULTANTS INC.
16 Young Street Woodstock, ON

LOGGED BY: J. Osborne
COMPLETION DEPTH: 25 Feet
REVIEWED BY: A. Rasoul
DRILL METHOD: SPT
DRILL DATE: May 31, 2021
PAGE: 1 of 1

PROJECT: Wyndcliffe Caledon	**BH LOCATION:** South corner of site	**BOREHOLE NO: BH6**
PROJECT NO: 6069	**LOCATION:** 14245 Highway 50, 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



Ground Surface

Topsoil

Silt and Clay
Medium brown, damp.

Silt and Clay
Medium brown, damp.

Silt and Clay
Medium brown, damp.

Silt and Clay
Medium brown, damp.

Grey, wet.

Silt and Clay
Grey, wet.

End of Log

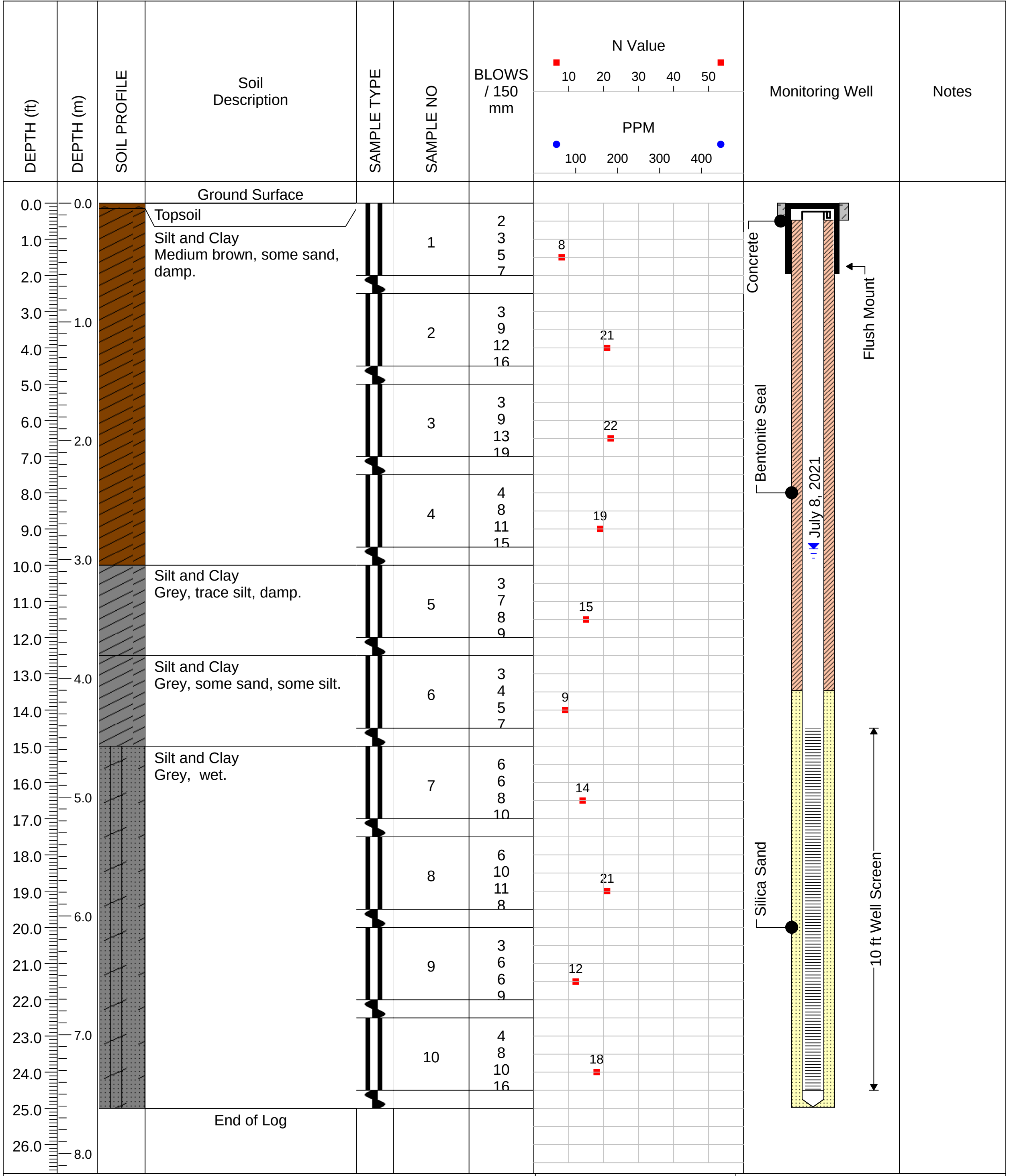
2 2 2 7	4
3 4 5 7	9
3 4 5 5	9
3 8 9 13	17
4 9 13 16	22
9 20 18 15	38
6 12 6 10	18
3 4 7 10	11
3 3 5 7	8
4 7 10 13	17

A & A ENVIRONMENTAL CONSULTANTS INC.
16 Young Street Woodstock, ON

LOGGED BY: J. Osborne	**COMPLETION DEPTH:** 25 Feet
REVIEWED BY: A. Rasoul	**DRILL METHOD:** SPT
DRILL DATE: June 7, 2021	**PAGE:** 1 of 1

PROJECT: Wyndcliffe Caledon	**BH LOCATION:** Northeast of HWY 50 entrance	**BOREHOLE NO: BH/MW7**
PROJECT NO: 6069	**LOCATION:** 14245 Highway 50, 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



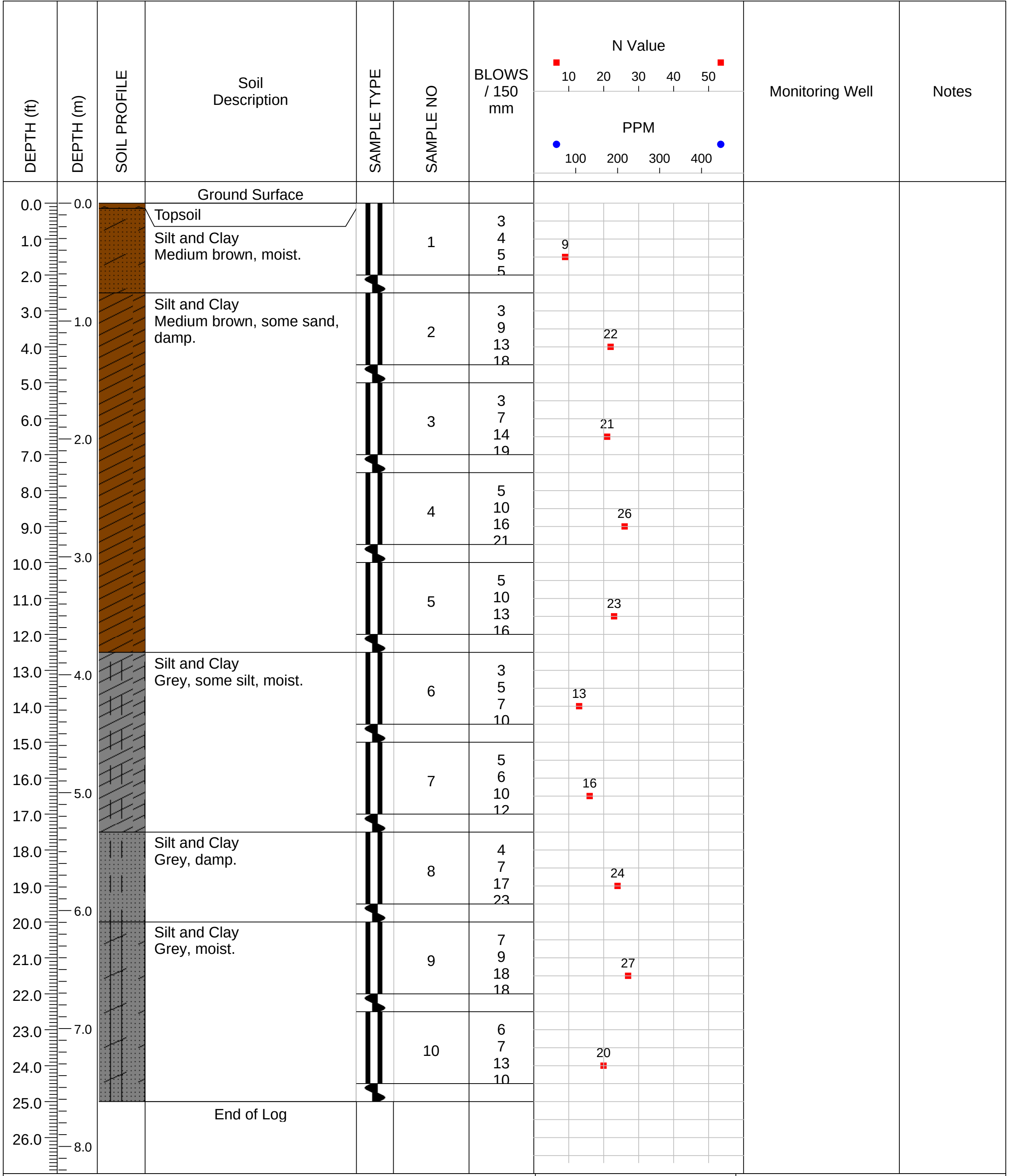
Soil Description	SAMPLE NO	BLOWS / 150 mm	N Value
Ground Surface			
Topsoil			
Silt and Clay Medium brown, some sand, damp.	1	2 3 5 7	8
	2	3 9 12 16	21
	3	3 9 13 19	22
	4	4 8 11 15	19
Silt and Clay Grey, trace silt, damp.	5	3 7 8 9	15
Silt and Clay Grey, some sand, some silt.	6	3 4 5 7	9
Silt and Clay Grey, wet.	7	6 6 8 10	14
	8	6 10 11 8	21
	9	3 6 6 9	12
	10	4 8 10 16	18
End of Log			

A & A ENVIRONMENTAL CONSULTANTS INC.
16 Young Street Woodstock, ON

LOGGED BY: J. Osborne	**COMPLETION DEPTH:** 25 Feet
REVIEWED BY: A. Rasoul	**DRILL METHOD:** SPT
DRILL DATE: June 7, 2021	**PAGE:** 1 of 1

PROJECT: Wyndcliffe Caledon	**BH LOCATION:** North of HWY 50 entrance	**BOREHOLE NO: BH8**
PROJECT NO: 6069	**LOCATION:** 14245 Highway 50, 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



Soil Description	Sample No	Blows / 150 mm	N Value
Ground Surface			
Topsoil			
Silt and Clay Medium brown, moist.	1	3 4 5 5	9
Silt and Clay Medium brown, some sand, damp.	2	3 9 13 18	22
	3	3 7 14 19	21
	4	5 10 16 21	26
	5	5 10 13 16	23
Silt and Clay Grey, some silt, moist.	6	3 5 7 10	13
	7	5 6 10 12	16
Silt and Clay Grey, damp.	8	4 7 17 23	24
Silt and Clay Grey, moist.	9	7 9 18 18	27
	10	6 7 13 10	20
End of Log			

A & A ENVIRONMENTAL CONSULTANTS INC.
16 Young Street Woodstock, ON

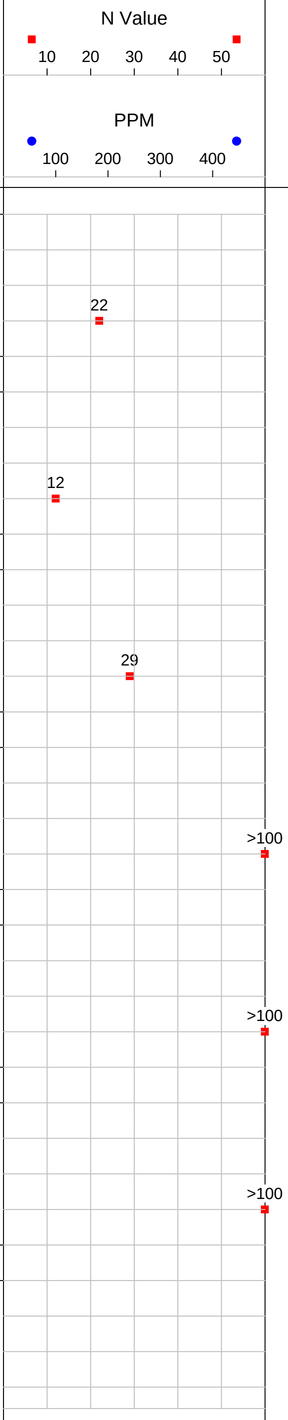
LOGGED BY: J. Osborne	**COMPLETION DEPTH:** 25 Feet
REVIEWED BY: A. Rasoul	**DRILL METHOD:** SPT
DRILL DATE: June 18, 2021	**PAGE:** 1 of 1

PROJECT: Wyndcliffe Caledon
BH LOCATION: Central north portion of site
BOREHOLE NO: BH9
PROJECT NO: 6069
LOCATION: 14245 Highway 50, 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	N Value	Monitoring Well	Notes
0.0	0.0	Ground Surface					
		Topsoil					
		Silt and Clay Medium brown, damp.	1	1 9 13 21	22		
			2	1 3 9 29	12		
			3	3 11 18 31	29		
		Silt and Clay Medium brown, trace sand, damp.	4	18 50-4	>100		
			5	15 50-4	>100		
			6	8 50-5	>100		
15.0		End of Log					



A & A ENVIRONMENTAL CONSULTANTS INC.
16 Young Street Woodstock, ON

LOGGED BY: J. Osborne
COMPLETION DEPTH: 15 Feet
REVIEWED BY: A. Rasoul
DRILL METHOD: SPT
DRILL DATE: June 23, 2021
PAGE: 1 of 1

PROJECT: Wyndcliffe Caledon | **BH LOCATION:** Central north portion of site | **BOREHOLE NO: BH10**

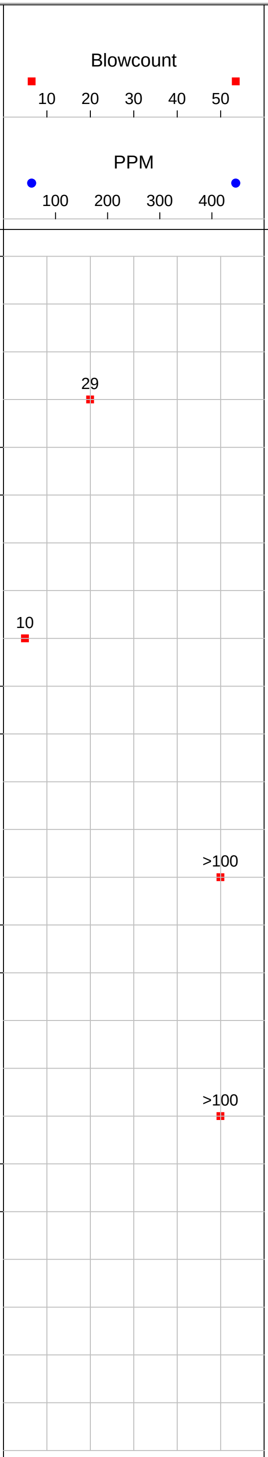
PROJECT NO: 6069 | **LOCATION:** 14245 Highway 50, Caledon ON

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	Monitoring Well	Notes
0.0	0.0	Ground Surface				
0.0	0.0	Topsoil				
0.3		Silt and Clay Medium brown, some sand, damp.				
0.0–2.0			1	3 9 20 24		
2.5–4.5			2	2 3 7 20		
5.0–7.0			3	3 17 50-5		
7.5–9.5			4	10 50-5		
10.0		End of Log				



A & A ENVIRONMENTAL CONSULTANTS INC.
16 Young Street Woodstock, ON

LOGGED BY: J. Osborne | **COMPLETION DEPTH:** 10 Feet

REVIEWED BY: A. Rasoul | **DRILL METHOD:** SPT

DRILL DATE: June 23, 2021 | **PAGE:** 1 of 1

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.

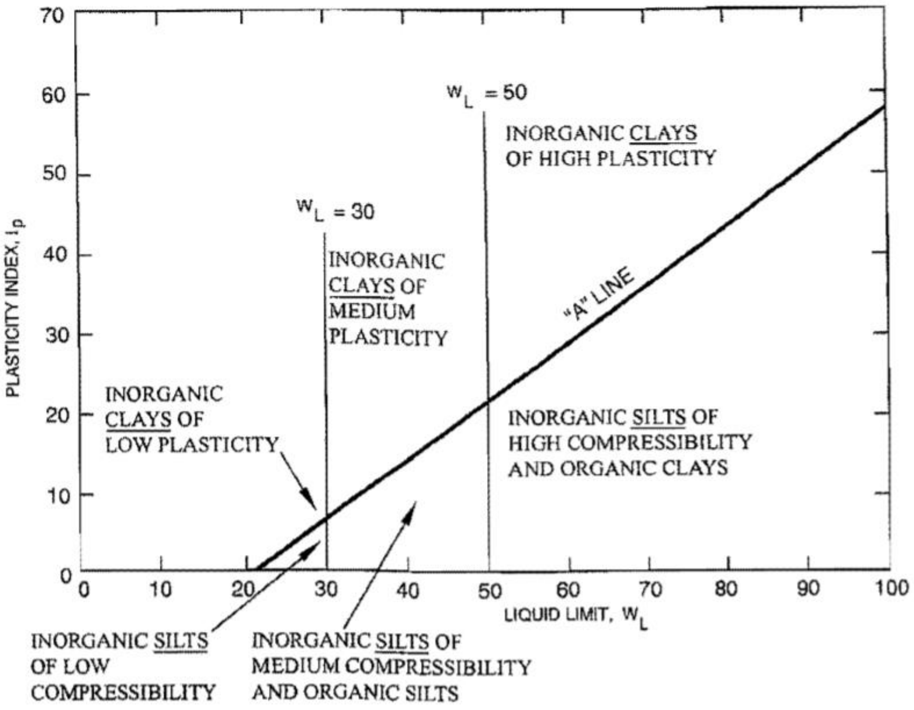
Abbreviations, graphic symbols and relevant test method designations are as follows:

w	Water Content
w_L, LL	Liquid Limit
w_p, PL	Plastic Limit
I_p	Plasticity Index
γ	Soil unit weight
K	Coefficient of Lateral earth pressure
K_s	Module of vertical subgrade reaction
p	Lateral earth pressure
q	Surcharge load
h	Depth from the ground surface
B	Width of rectangular footing
P	Hydrostatic uplift pressure
d	Depth of structure's base below the design water level
γ_w	Unit weight of water
Φ	Geotechnical resistance factor
ϕ	Internal friction angle of soil
c	Cohesion
c_u, S_u	Undrained shear strength
V_s	Shear wave velocity
SPT-N	Penetration resistance
SPMMD	Standard Proctor Maximum Dry Density
MRD	Marshal Maximum Relative Density

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

noun	gravel, sand, silt, clay	**> 35 % and main fraction**
"and"	and gravel, and silt, etc.	**>35 %**
adjective	gravelly, sandy, silty, clayey, etc.	**20 to 35 %**
"some"	some sand, some silt, etc.	**10 to 20%**
"trace"	trace sand, trace silt, etc.	**1 to 10 %**

The plasticity chart (after Casagrande, 1948):



Correlation of soil parameters with uncorrected SPT values for: a) cohesionless soils and b) cohesive soil

Compactness Condition	SPT N-INDEX (blows per 0.3 m)
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

(a)

Consistency	Undrained Shear Strength (kPa)	SPT N-INDEX (blows per 0.3 m)
Very soft	< 12	0 to 2
Soft	12 - 25	2 to 4
Firm	25-50	4 to 8
Stiff	50 - 100	8 to 15
Very stiff	100 - 200	15 to 30
Hard	>200	>30

(b)

- *Standard Penetration Tests (SPT); followed the methods described in ASTM Standard D1586-08a. The number of blows by a 63.5 kg (140 lb) hammer dropped from 760 mm (30 in.) is recorded for a depth of 460 mm (18”). The last two 150 mm distances (total = 300 mm) are used to calculate the SPT-N index.*

APPENDIX C – Grain Size Distribution and Test Results

Orbit Engineering Limited
1900 Clark Boulevard, Unit 9
Brampton, ON, L6T 0E9
Tel: +1 905 494 0074
Fax: +1 855 666 3355
www.orbitengineering.ca,
info@orbitengineering.ca

GEOTECHNICAL TESTING REPORT DATA

6069 – WYNDCLIFFE CALEDON, ON

Prepared for:

A & A Environmental Consultants Inc.

By:

Orbit Engineering Limited

Project No. OE201046AG

July 23, 2021

ORBIT ENGINEERING
Consulting Engineers

Orbit Engineering Limited
1900 Clark Boulevard, Unit 9
Brampton, ON, L6T 0E9
Tel: +1 905 494 0074
Fax: +1 855 666 3355
www.orbitengineering.ca,
info@orbitengineering.ca

July 23, 2021

A&A Environmental Consultants
16 Young Street
Woodstock, Ontario
N4S 3L4
Email: mheidari@aaenvironmental.ca

Attention: **Dr. Ali A. Rasoul Ph.D, EP, P.Geo, QP - Principle**

RE: **LABORATORY TEST RESULTS - Project: 6069 – Wyndcliffe Caledon, ON**

Dear Mr. Rasoul,

Orbit Engineering Limited (Orbit) is pleased to provide the Final LABORATORY TESTING REPORT DATA for the above-mentioned project. The report presents the results of laboratory testing carried out on soil samples received at Orbit Laboratory on June 25th, 2021.
The laboratory testing included the following:

1. Water Moisture Content - ASTM D2216
2. Particle Size Analysis (Hydrometer) - ASTM D422 - D2217
3. Atterberg Limits - ASTM 4318

The results of the testing are summarized in the attached Table 1 and details of testing results are shown in Appendix **A**.

We trust that this information meets your present requirements. If we can be of additional assistance in this regard, please contact this office.

For and on behalf of Orbit Engineering Limited,

Aly Ahmed

Aly Ahmed, Ph D, P.Eng.,
Lab Supervisor

Hafiz Muneeb Ahmad, M.Sc.,P.Eng.,
Principal Engineer
Professional Supervising Engineer

Table 1: Summary of Laboratory Testing Results (A & A Project: 6069–Wyndcliffe Caledon, ON)

Sample No.	Depth (ft)	Water Content (%)	Atterberg Limits (%)			Soil Compositions (%)				Soil Description
			LL	PL	PI	Gravel	Sand	Silt	Clay	
BH1	55-57	15.2	24.4	14.1	10.3	1	14	64	21	Clayey Silt, some Sand trace Gravel
BH2	32.5-34.5	18.9	20.5	13.3	7.2	1	2	84	13	Silt, some Clay trace Sand and Gravel
BH4	42.5-44.5	28.7	37.7	18.5	19.2	-	1	57	42	Silt and Clay, trace Sand
BH7	22.5-24.5	14.3	Non-Plastic			4	36	52	8	Silt and Sand, trace Clay and Gravel
BH8	10-12	21.2	41.2	19.5	21.7	1	6	49	44	Silt and Clay, trace Sand and Gravel

CLOSURE

We trust that this information is satisfactory for your present requirements. Should you have any questions or require additional information, please do not hesitate to contact this office.

For and Behalf of Orbit Engineering Limited,

Ameer Rizvi, B.Sc.
Lab Technician

Aly Ahmed

Aly Ahmed, Ph D., P.Eng.
Lab Supervisor

Reviewed by:

Hafiz Muneeb Ahmad, M.Sc.,P.Eng.
Principal Engineer
Professional Supervising Engineer

ORBITENGINEERING

NATURAL MOISTURE CONTENT

LABORATORY SERVICES

LS - 703

Project No.: OE201046AG

Lab Technician: Ameer Rizvi

Borehole No. BH1,BH2,BH4,BH7&BH8

Date 8-Jul-21

NATURAL MOISTURE CONTENT

Sample No.	Weight of tare (g)	Weight of tare + soil (g)	Weight of tare + soil (dry) (g)	Moisture content (%)
BH1 @ 55-57	6.42	392.29	341.34	**15.2**
BH2 @ 32.5-34.5	6.42	389.07	328.14	**18.9**
BH4 @ 42.5-44.5	6.43	391.90	306.00	**28.7**
BH7 @ 22.5-24.5	6.53	399.85	350.59	**14.3**
BH8 @ 10-12	6.60	399.16	330.39	**21.2**

Aly Ahmed

Aly Ahmed, Ph D., P Eng
Lab Supervisor

ORBIT ENGINEERING

ATTERBERG LIMITS

LABORATORY SERVICES

(LS-703, 704/D4318)

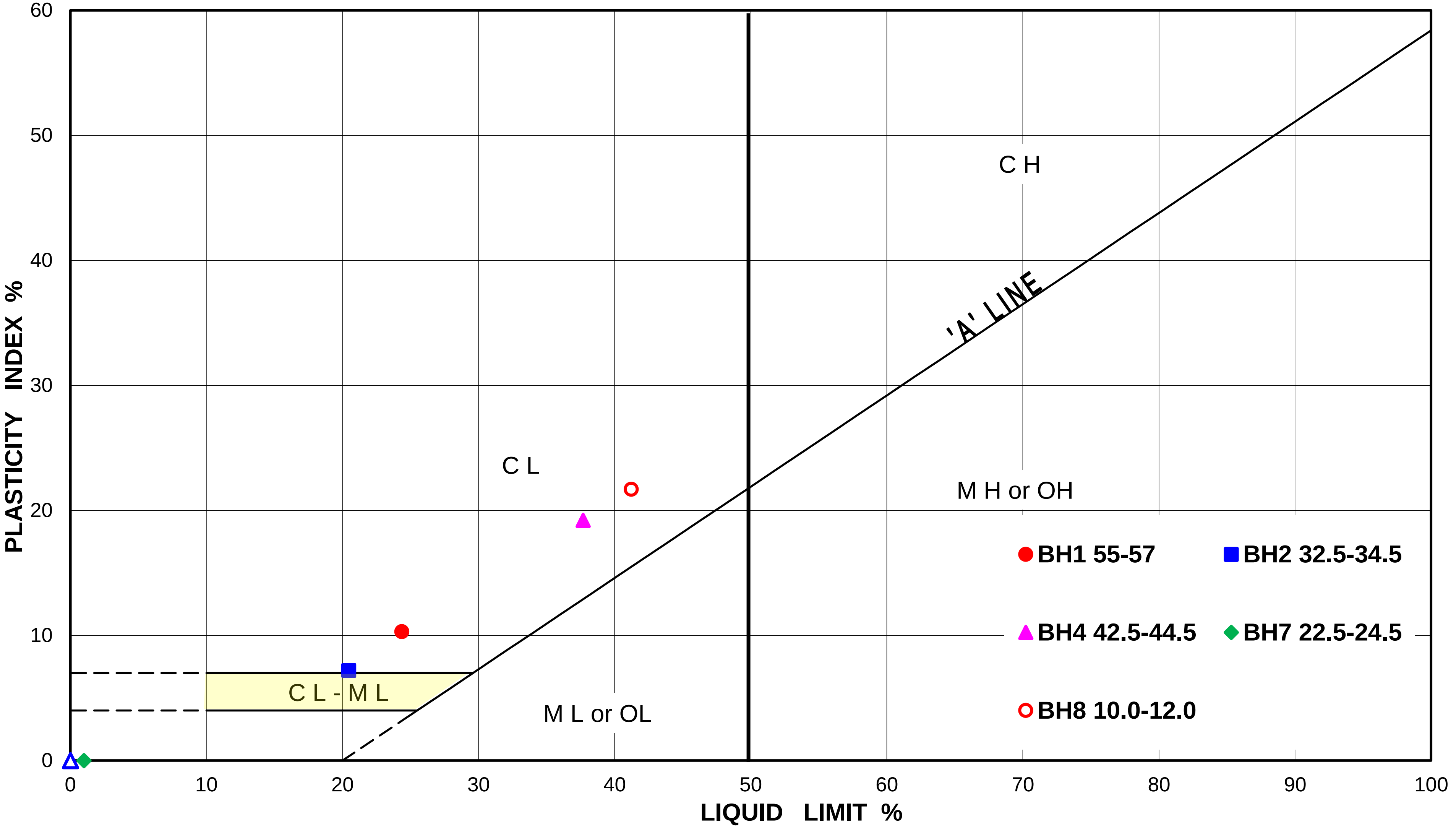
PLASTICITY CHART WORKSHEET

Date :	**23-Jul-21**	**Lab Number :**	**1078**
Project Number :	**OE201046AG**	**Figure Number :**	**A0**
Drawing Number :	**B3**		
Soil Description :			

Borehole Number	Sample Type	Sample Number	Natural MC (%)	Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Liquidity Index (%)
BH1	CL	BH1-S1	15.20	55-57	24.40	14.10	10.30	0.11
BH2	CL	BH2-S2	18.90	32.5-34.5	20.50	13.30	7.20	0.78
BH4	CL	BH4-S3	28.70	42.5-44.5	37.70	18.50	19.20	0.53
BH7	Non-Plastic	BH7-S4	14.30	22.5-24.5	Non-Plastic	Non-Plastic	Non-Plastic	Non-Plastic
BH8	CL-ML	BH8-S5	21.20	10-12	41.20	19.50	21.70	0.08

Aly Ahmed

Aly Ahmed, Ph D., P Eng
Lab Supervisor



ORBIT ENGINEERING

PLASTICITY CHART

Project No:	OE201046AG
Drawing No.	B15
Date:	22-Jul-21

ORBIT ENGINEERING

ATTERBERG LIMITS

(LS-703, 704/D4318)

LABORATORY SERVICES

Project No.: OE201046AG

Operator: Ameer Rizvi

Borehole N BH1

Date: 22-Jul-21

LIQUID LIMIT									
SAMPLE NO.	55-57								
NO OF BLOWS	35	26	22						
DISH NO	1	2	3						
DISH + WET SOIL	32.71	33.7	33.41						
DISH + DRY SOIL	30.52	31.32	30.95						
MOISTURE	2.19	2.38	2.46						
DISH	21.19	21.26	21.17						
DRY SOIL	9.33	10.06	9.78						
% MOISTURE	23.47	23.66	25.15						
PLASTIC LIMIT									
DISH NO	4	5							
DISH + WET SOIL	31.1	30.4							
DISH + DRY SOIL	29.9	29.24							
MOISTURE	1.2	1.16							
DISH	21.24	21.13							
DRY SOIL	8.66	8.11							
% MOISTURE	13.86	14.30							
PLASTIC LIMIT %	14.1								
LIQUID LIMIT %	24.4								
PLASTICITY INDEX	10.3								

Aly Ahmed

Aly Ahmed, Ph D., P Eng

Lab Supervisor

ORBITENGINEERING

ATTERBERG LIMITS

(LS-703, 704/D4318)

LABORATORY SERVICES

Project No.: OE201046AG

Operator: Ameer Rizvi

Borehole N BH2

Date 22-Jul-21

LIQUID LIMIT									
SAMPLE NO.	32.5-34.5								
NO OF BLOWS	35	30	20						
DISH NO	6	7	8						
DISH + WET SOIL	33.4	33.58	34.51						
DISH + DRY SOIL	31.41	31.51	32.2						
MOISTURE	1.99	2.07	2.31						
DISH	21.22	21.24	21.18						
DRY SOIL	10.19	10.27	11.02						
% MOISTURE	19.53	20.16	20.96						
PLASTIC LIMIT									
DISH NO	9	10							
DISH + WET SOIL	30.3	31.1							
DISH + DRY SOIL	29.21	29.93							
MOISTURE	1.09	1.17							
DISH	21.08	21.04							
DRY SOIL	8.13	8.89							
% MOISTURE	13.41	13.16							
PLASTIC LIMIT %	13.3								
LIQUID LIMIT %	20.5								
PLASTICITY INDEX	7.2								

Aly Ahmed

Aly Ahmed, Ph D., P Eng
Lab Supervisor

ORBIT ENGINEERING

ATTERBERG LIMITS

(LS-703, 704/D4318)

LABORATORY SERVICES

Project No.: OE201046AG

Operator: Ameer Rizvi

Borehole N BH4

Date: 22-Jul-21

LIQUID LIMIT									
SAMPLE NO.	42.5-44.5								
NO OF BLOWS	33	27	21						
DISH NO	11	12	13						
DISH + WET SOIL	33.56	34.2	33.72						
DISH + DRY SOIL	30.2	30.64	30.28						
MOISTURE	3.36	3.56	3.44						
DISH	21.07	21.17	21.26						
DRY SOIL	9.13	9.47	9.02						
% MOISTURE	36.80	37.59	38.14						
PLASTIC LIMIT									
DISH NO	14	15							
DISH + WET SOIL	29.85	30.15							
DISH + DRY SOIL	28.5	28.72							
MOISTURE	1.35	1.43							
DISH	21.17	21.01							
DRY SOIL	7.33	7.71							
% MOISTURE	18.42	18.55							
PLASTIC LIMIT %	18.5								
LIQUID LIMIT %	37.7								
PLASTICITY INDEX	19.2								

Aly Ahmed

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Lab Supervisor

ORBIT ENGINEERING

ATTERBERG LIMITS

(LS-703, 704/D4318)

LABORATORY SERVICES

Project No.: OE201046AG

Operator Ameer Rizvi

Borehole N(BH7

Date 22-Jul-21

LIQUID LIMIT									
SAMPLE NO.	22.5-24.5								
NO OF BLOWS									
DISH NO	16	17	18						
DISH + WET SOIL									
DISH + DRY SOIL									
MOISTURE	0	0	0						
DISH	21.1	21.07	21.19						
DRY SOIL	-21.1	-21.07	-21.19						
% MOISTURE	0.00	0.00	0.00						
PLASTIC LIMIT									
DISH NO	19	20							
DISH + WET SOIL									
DISH + DRY SOIL									
MOISTURE	0	0							
DISH	21.25	21							
DRY SOIL	-21.25	-21							
% MOISTURE	0.00	0.00							
PLASTIC LIMIT %	Non-Plastic								
LIQUID LIMIT %	Non-Plastic								
PLASTICITY INDEX	Non-Plastic								

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ORBIT ENGINEERING

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LABORATORY SERVICES

Project No.: OE201046AG

Operator: Ameer Rizvi

Borehole N BH8

Date: 22-Jul-21

LIQUID LIMIT									
SAMPLE NO.	10.0-12.0								
NO OF BLOWS	34	29	20						
DISH NO	16	17	18						
DISH + WET SOIL	33.25	34.45	34.62						
DISH + DRY SOIL	29.75	30.57	30.65						
MOISTURE	3.5	3.88	3.97						
DISH	21.01	21.07	21.19						
DRY SOIL	8.74	9.5	9.46						
% MOISTURE	40.05	40.84	41.97						
PLASTIC LIMIT									
DISH NO	19	20							
DISH + WET SOIL	30.21	29.67							
DISH + DRY SOIL	28.75	28.25							
MOISTURE	1.46	1.42							
DISH	21.25	21							
DRY SOIL	7.5	7.25							
% MOISTURE	19.47	19.59							
PLASTIC LIMIT %	19.5								
LIQUID LIMIT %	41.2								
PLASTICITY INDEX	21.7								

Aly Ahmed

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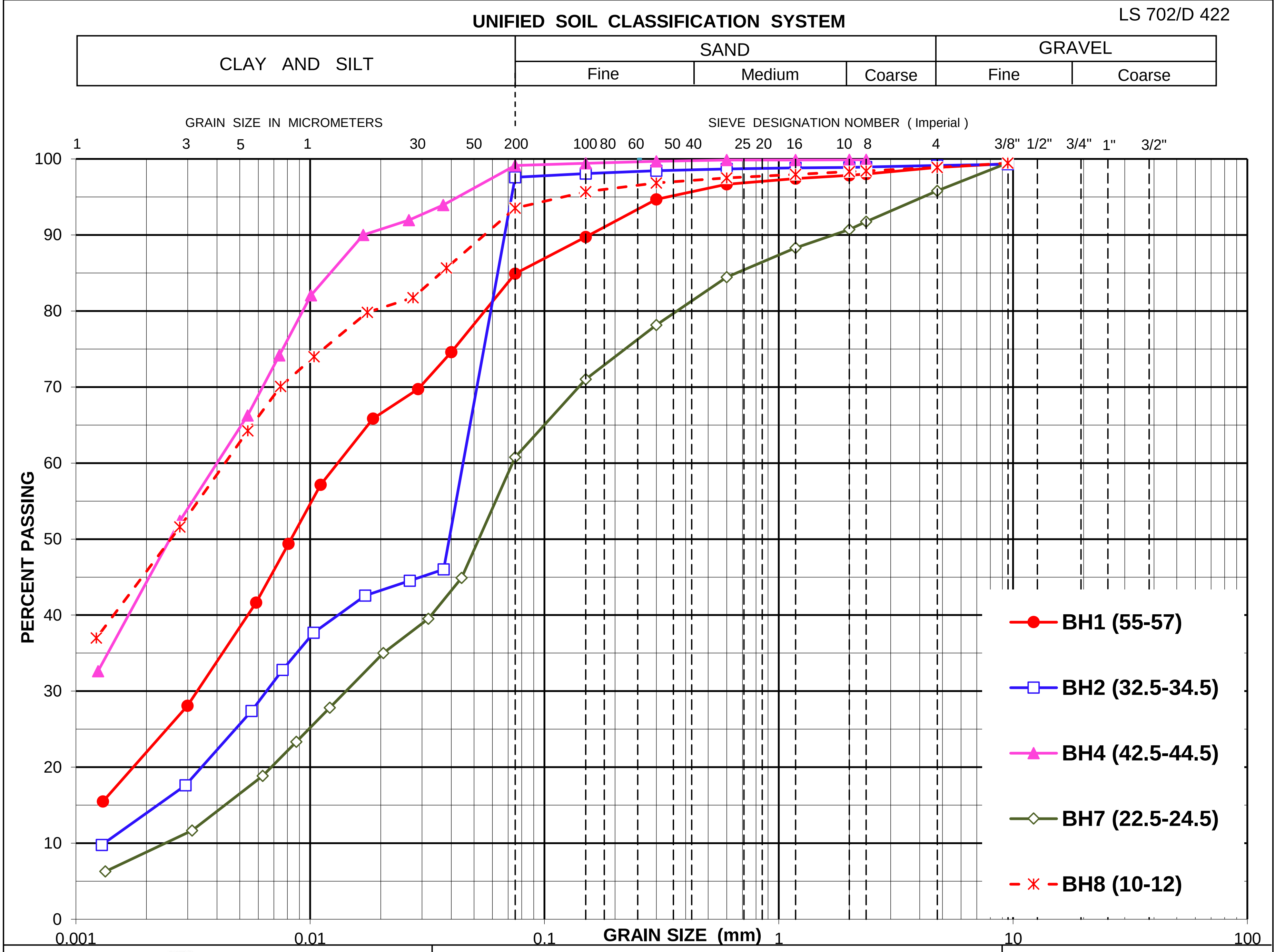
Lab Supervisor



ORBITENGINEERING
Consulting Engineers

GRAIN SIZE DISTRIBUTION

Figure No.:	**B1**
PROJECT No. :	**OE201046AG**
DATE :	**July 12, 2021**