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A REPORT TO
TOWN OF NEWMARKET

A GEOTECHNICAL INVESTIGATION FOR AN EXISTING RESIDENTIAL PROPERTY

16780 YONGE STREET

TOWN OF NEWMARKET

REFERENCE NO. 1708-S019

OCTOBER 2017

DISTRIBUTION

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1.0 INTRODUCTION

In accordance with written authorization dated August 1, 2017, from Ms. Sepideh Majdi, of Town of Newmarket, a geotechnical investigation was carried out at 16780 Yonge Street, in the Town of Newmarket, for an existing residential property.

The purpose of the investigation was to reveal the subsurface conditions and to determine the engineering properties of the disclosed soils.

The findings and resulting geotechnical recommendations are presented in this Report.

2.0 **SITE AND PROJECT DESCRIPTION**

The Town of Newmarket is situated on Schomberg Lake (glacial) plain where drift has been partly eroded and filled, in places, with lacustrine clay, silt and sand.

The investigated site is situated at the northwest corner of Mulock Drive and Yonge Street, in the Town of Newmarket. The site area is occupied by a residential home and the area is generally grass-covered with trees. The ground surface is relatively flat and level, with minor undulations.

3.0 FIELD WORK

The geotechnical field work, consisting of 5 boreholes (Borehole Nos. 1, 6, 7, 8 and 9) to depths of 6.6 m and 6.7 m, was performed on September 1, 2017, at the locations shown on the Borehole Location Plan, Drawing No. 1. One monitoring well was installed at Borehole 1 for future groundwater monitoring.

The holes were advanced at intervals to the sampling depths by a truck-mounted, continuous-flight power-auger machine equipped for soil sampling. Standard Penetration Tests, using the procedures described on the enclosed “List of Abbreviations and Terms”, were performed at the sampling depths. The test results are recorded as the Standard Penetration Resistance (or ‘N’ values) of the subsoil. The relative density of the granular strata and the consistency of the cohesive strata are inferred from the ‘N’ values. Split-spoon samples were recovered for soil classification and laboratory testing.

The field work was supervised and the findings were recorded by a Geotechnical Technician.

The sampling depths and the depths of the soil strata changes were referred to the prevailing ground surface at each of the borehole locations.

4.0 **SUBSURFACE CONDITIONS**

Detailed descriptions of the encountered subsurface conditions are presented on the Borehole Logs, comprising Figures 1 to 5, inclusive. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing No. 2, and the engineering properties of the disclosed soils are discussed herein.

This investigation has disclosed that beneath a veneer of topsoil, the site is generally underlain by strata of silty clay, silt and sandy silt.

4.1 **Topsoil** (All Boreholes)

The revealed topsoil ranges from 25 to 43 cm thick. It is dark brown in colour, indicating that it contains appreciable amounts of roots and humus. These materials are unstable and compressible under loads; therefore, the topsoil is considered to be void of engineering value. Due to its humus content, it may produce volatile gases and generate an offensive odour under anaerobic conditions. Therefore, the topsoil must not be buried below any structures or deeper than 1.2 m below the finished grade, so it will not have an adverse impact on the environmental well-being of the developed areas.

Since the topsoil is considered void of engineering value, it can only be used for general landscaping and landscape contouring purposes. A fertility analysis can determine the suitability of the topsoil as a planting material. In places, thicker topsoil than that revealed by the boreholes may occur.

4.2 **Silty Clay** (Boreholes 6, 8 and 9)

The silty clay stratum was encountered beneath a layer of silt and extends to the maximum investigated depth at all above boreholes. The clay is laminated with wet sand and silt seams and layers, showing that it is a lacustrine deposit.

The obtained ‘N’ values range from 16 to 31, with a median of 25 blows per 30 cm of penetration, indicating that the consistency of the clay is stiff to very stiff, being generally very stiff.

The Atterberg Limits of 2 representative samples and the water content of the samples were determined. The results are plotted on the Borehole Log and summarized below:

Liquid Limit	29% and 30%
Plastic Limit	16% and 17%
Natural Water Content	17% to 20% (median 19%)

The above results show that the clay is a cohesive material with low plasticity. The natural water content values generally lie slightly above its plastic limits, confirming the generally very stiff consistency of the clay as determined from the ‘N’ values.

Grain size analyses were performed on 2 samples of the silty clay; the results are plotted on Figure 6.

Based on the above findings, the following engineering properties are deduced:

- High frost susceptibility and high soil-adfreezing potential.
- Low water erodibility.

- Low permeability, with an estimated coefficient of permeability of 10^{-7} cm/sec, an estimated percolation rate of over 80 min/cm, and runoff coefficients of:

Slope	
0% - 2%	0.15
2% - 6%	0.20
6% +	0.28

- A cohesive-frictional soil, its shear strength is derived from consistency and augmented by the internal friction of the silt. Its shear strength is moisture dependent.
- The clay will be prone to sloughing if it is exposed for prolonged periods in steep cuts. This would generally be initiated by infiltrating precipitation or groundwater seeping out from the silt and fine sand layers.
- A very poor pavement-supportive material, with an estimated California Bearing Ratio (CBR) value of 3% or less.
- Moderately high corrosivity to buried metal, with an estimated electrical resistivity of 2500 ohm·cm.

4.3 **Silt** and **Sandy Silt** (All Boreholes)

The silt deposits were encountered at various depths and extends to the maximum depth at Boreholes 1 and 7. The silt is laminated with seams and layers of silty clay and fine sand. The laminated structure shows that the silt is a glaciolacustrine deposit.

The natural water content values of the silts range from 6% to 24%, with a median of 19%, indicating it is in a moist to wet condition. The wet samples are water bearing and became highly dilatant when shaken by hand.

The obtained ‘N’ values range from 3 to 65, with a median of 20 blows per 30 cm of penetration, indicating that the relative density of the silts is loose to very dense, being generally compact.

A grain size analysis was performed on 1 representative sample of the silt and the result is plotted on Figure 7.

Based on the above findings, the engineering properties relating to the project are given below:

- Highly frost susceptible, with high soil-adfreezing potential.
- Highly water erodible; they are susceptible to migration through small openings under seepage pressure.
- Relatively low permeability, with an estimated coefficient of permeability of 10^{-6} cm/sec, an estimated percolation rate of 30 min/cm, depending on the clay content, and runoff coefficients of:

Slope	
0% - 2%	0.15
2% - 6%	0.20
6% +	0.28

- The soil has a high capillarity and water retention capacity.
- A frictional soil, its shear strength is density dependent. Due to the dilatancy, the strength of the wet silt is susceptible to impact disturbance; i.e., the disturbance will induce a build-up of pore pressure within the soil mantle, resulting in soil dilation and a reduction in shear strength.
- In excavation, the moist silt will be stable in relatively steep cuts, while the wet silt will slough and run slowly with seepage bleeding from the cut face, and the bottom will boil under a piezometric head of 0.3 m.

- A poor pavement-supportive material, with an estimated CBR value of 6%.
- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 5000 ohm·cm.

4.4 Compaction Characteristics of the Revealed Soils

The obtainable degree of compaction is primarily dependent on the soil moisture and, to a lesser extent, on the type of compactor used and the effort applied.

As a general guide, the typical water content values of the revealed soils for Standard Proctor compaction are presented in Table 1.

Table 1 - Estimated Water Content for Compaction

Soil Type	**Determined Natural Water Content (%)**	**Water Content (%) for Standard Proctor Compaction**	
		100% (optimum)	**Range for 95% or +**
Silty Clay	17 to 20 (median 19)	18	14 to 23
Silt and Sandy Silt	6 to 24 (median 19)	11 to 13	7 to 17

Based on the above findings, the silty clay is generally suitable for a 95% or + Standard Proctor compaction. However, the silts are generally too wet and will require aeration or mixing with drier soils prior to structural compaction.

The silty clay should be compacted using a heavy-weight, kneading-type roller. The silts can be compacted by a smooth roller with or without vibration, depending on the water content of the soil being compacted. The lifts for compaction should be limited to 20 cm, or to a suitable thickness as assessed by test strips performed by the equipment which will be used at the time of construction.

It is difficult to monitor the lifts of backfill placed in deep trenches; therefore, it is preferable that the compaction of backfill at depths over 1.0 m below the pavement subgrade be carried out on the wet side of the optimum. This would allow a wider latitude of lift thickness.

One should be aware that with considerable effort, a 90%± Standard Proctor compaction of the wet silts is achievable. Further densification is prevented by the pore pressure induced by the compactive effort; however, large random voids will have been expelled and, with time, the pore pressure will dissipate and the percentage of compaction will increase. There are many cases on record where after a few months of rest, the density of the compacted mantle has increased to over 95% of its maximum Standard Proctor dry density.

If the compaction of the soils is carried out with the water content within the range for 95% Standard Proctor dry density but on the wet side of the optimum, the surface of the compacted soil mantle will roll under the dynamic compactive load. This is unsuitable for pavement construction since each component of the pavement structure is to be placed under dynamic conditions which will induce the rolling action of the subgrade surface and cause structural failure of the new pavement. The foundation or bedding of the sewer and slab-on-grade will be placed on a subgrade which will not be subjected to impact loads. Therefore, the structurally compacted soil mantle with the water content on the wet side or dry side of the optimum will provide an adequate subgrade for the construction.

5.0 GROUNDWATER CONDITIONS

Groundwater seepage encountered during augering was recorded on the field logs. The level of groundwater and/or the occurrence of cave-in were measured upon completion of the boreholes; the data are plotted on the Borehole Logs and listed in Table 2.

Table 2 - Groundwater Levels

		Soil Colour Changes Brown to Grey	**Seepage Encountered During Augering**		**Measured Groundwater/ Cave-In* Level On Completion**
BH No.	**Borehole Depth (m)**	**Depth (m)**	**Depth (m)**	**Amount**	**Depth (m)**
1	6.7	3.8	0.8	Some	5.8
6	6.6	4.0	0.5	Some	Dry
7	6.6	6.6+	0.5	Some	Dry
8	6.6	4.0	0.5	Some	Dry
9	6.6	4.0	0.4	Slight	4.3

Groundwater was measured at depths of 5.8 m and 4.3 m below the prevailing ground surface at Boreholes 1 and 9, respectively. The remaining boreholes were dry upon completion of the field work. The groundwater level will be subject to seasonal fluctuations.

The soil colour changes from brown to grey at depths of 3.8 m and 4.0 m below the prevailing ground surface, indicating that the soils in the upper zone have oxidized. The encountered groundwater levels generally represent the groundwater regime of the site at the time of the investigation, and the groundwater regime is subject to seasonal fluctuation.

The groundwater yield from the silty clay, due to its low permeability, is expected to be small and limited. However, in the strata of silt, the groundwater yield is expected to be moderate.

6.0 **DISCUSSION AND RECOMMENDATIONS**

The investigation has disclosed that beneath a veneer of topsoil, the site is underlain by strata of stiff to very stiff, generally very stiff silty clay; and loose to very dense, generally compact silt and sandy silt. The surficial soils are weathered to depths of 0.7 m and 1.4 m below the prevailing ground surface.

Groundwater was measured at depths of 5.8 m and 4.3 m below the prevailing ground surface at Boreholes 1 and 9, respectively. The remaining boreholes were dry upon completion of the field work. The groundwater level will be subject to seasonal fluctuations.

The groundwater yield from the silty clay, due to its low permeability, is expected to be small and limited. However, in the strata of silts, the groundwater yield is expected to be moderate.

The geotechnical findings which warrant special consideration are presented below:

1. The topsoil must be stripped for the project construction. This material is unsuitable for engineering applications; therefore, it should be placed in the landscaped areas only and should not be buried below any structures, or deeper than 1.2 m below the exterior finished grade of the project.
2. The sound natural soils are suitable for normal spread and strip footing construction. However, due to the presence of topsoil and weathered soil, the footing subgrade must be inspected by either a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to ensure that its condition is compatible with the design of the foundations.

3. For slab-on-grade construction, any weathered, soft or loose soils should be subexcavated, aerated and properly compacted prior to the placement of the slab. Any new material for raising the grade should consist of organic-free soil compacted to at least 98% of its maximum Standard Proctor dry density. The slab should be constructed on a granular base, 20 cm thick, consisting of 20-mm Crusher-Run Limestone, or equivalent, compacted to its maximum Standard Proctor dry density.
4. A Class ‘B’ bedding, consisting of compacted 20-mm Crusher-Run Limestone, is recommended for the construction of the underground services. Where water-bearing silt occurs, the sewer joints should be leak-proof, or wrapped with an appropriate waterproof membrane to prevent subgrade migration.
5. The revealed soils are highly frost susceptible, with high soil-adfreezing potential. Where they are used to backfill against foundation walls, special measures must be incorporated into the building construction to prevent serious damage due to soil adfreezing.

The recommendations appropriate for the project described in Section 2.0 are presented herein. One must be aware that the subsurface conditions may vary between boreholes. Should this become apparent during construction, a geotechnical engineer must be consulted to determine whether the following recommendations require revision.

6.1 **Foundations**

Based on the borehole findings, the recommended soil pressures and suitable founding levels are presented in Table 3.

Table 3 - Founding Levels

	Recommended Maximum Allowable Soil Pressure (SLS)/ Factored Ultimate Soil Bearing Pressure (ULS) and Suitable Founding Level			
	70 kPa (SLS) 110 kPa (ULS)	**150 kPa (SLS) 250 kPa (ULS)**	**200 kPa (SLS) 320 kPa (ULS)**	**300 kPa (SLS) 480 kPa (ULS)**
BH No.	**Depth (m)**	**Depth (m)**	**Depth (m)**	**Depth (m)**
1	1.0 or +	-	3.8 or +	5.4 or +
6	-	1.0 or +	1.6 or +	6.2 or +
7	1.0 or +	1.6 or +	2.4 or +	6.2 or +
8	-	-	1.0 or +	2.4 or +
9	-	1.0 or +	2.4 or +	-

Where earth fill is required to raise the site, it is generally more economical to place the fill in an engineered manner for normal footing construction. The requirements for engineered fill construction are discussed in Section 6.2.

The recommended soil pressure (SLS) incorporates a safety factor of 3. The total and differential settlements of the footings are estimated to be 25 mm and 15 mm, respectively.

The foundations exposed to weathering and in unheated areas should have at least 1.2 m of earth cover for protection against frost action, or must be properly insulated.

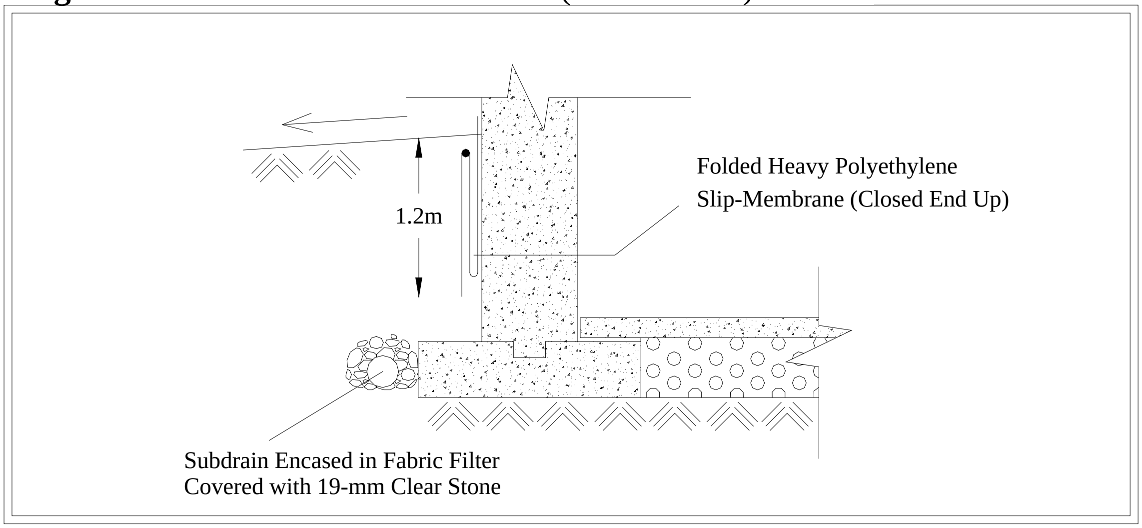
To ensure that the condition of the subgrade is compatible with the foundation design requirements, the footing subgrade of the normal foundations must be inspected by a geotechnical engineer, or a senior technician under the supervision of a geotechnical engineer.

If higher bearing is required due to the existing soil condition, further investigation with deeper boreholes will be required for deep foundation recommendations, such as caissons or piles.

Where a basement is contemplated, perimeter subdrains and dampproofing of the foundation walls will be required. All the subdrains must be encased in a fabric filter to protect them against blockage by silting, and must be connected to a positive outlet.

The occurring soils are high in frost heave and soil-adfreezing potential. If these soils are to be used for the foundation backfill, the foundation walls should be shielded by a polyethylene slip-membrane for protection against soil adfreezing. The membrane will allow vertical movement of the heaving soil (due to frost) without imposing structural distress on the foundations. The recommended measures are schematically illustrated in Diagram 1.

Diagram 1 - Frost Protection Measures (Foundations)



The necessity to implement the above recommendations should be further assessed by a geotechnical engineer at the time of construction.

The design of the foundations should meet the requirements specified in the latest Ontario Building Code, and the structure should be designed to resist an earthquake force using Site Classification ‘D’ (stiff soil).

6.2 **Engineered Fill**

Where earth fill is required to raise the site, or where extended footings are necessary, the engineering requirements for a certifiable fill for road construction, municipal services, slab-on-grade, and footings designed with a Maximum Allowable Soil Pressure (SLS) of 150 kPa and a Factored Ultimate Soil Bearing Pressure (ULS) of 250 kPa for normal footings are presented below:

1. The topsoil must be removed.
2. The highly weathered soils must be subexcavated, and the subgrade must be inspected and proof-rolled prior to any fill placement.
3. Inorganic soils must be used, and they must be uniformly compacted in lifts 20 cm thick to 98% or + of their maximum Standard Proctor dry density up to the proposed finished grade. The soil moisture must be properly controlled on the wet side of the optimum.
4. If imported fill is to be used, it should be inorganic soils, free of deleterious material with environmental issue (contamination). Any potential imported earth fill from off site must be reviewed for geotechnical and environmental quality by the appropriate personnel as authorized by the developer or agency, before it is hauled to the site.
5. If foundations are to be built soon after the fill placement, the densification process for the engineered fill must be increased to 100% of the maximum Standard Proctor compaction.
6. If the engineered fill is to be left over the winter months, adequate earth cover or equivalent must be provided for protection against frost action.
7. The engineered fill must extend over the entire graded area; the engineered fill envelope and finished elevations must be clearly and accurately defined in the field, and they must be precisely documented by qualified surveyors.

Foundations partially on engineered fill must be reinforced by two 15-mm steel reinforcing bars in the footings and upper section of the foundation walls, or be designed by a structural engineer, to properly distribute the stress induced by the abrupt differential settlement (about 15 mm) between the natural soil and engineered fill.

8. The engineered fill must not be placed during the period from late November to early April when freezing ambient temperatures occur either persistently or intermittently. This is to ensure that the fill is free of frozen soils, ice and snow.
9. Where the fill is to be placed on a bank steeper than 1 vertical:3 horizontal, the face of the bank must be flattened to 3 + so that it is suitable for safe operation of the compactor and the required compaction can be obtained.
10. Where the ground is wet due to subsurface water seepage, an appropriate subdrain scheme must be implemented prior to the fill placement, particularly if it is to be carried out on sloping ground.
11. The fill operation must be inspected on a full-time basis by a technician under the direction of a geotechnical engineer.
12. The footing and underground services subgrade must be inspected by the geotechnical consulting firm that supervised the engineered fill placement. This is to ensure that the foundations are placed within the engineered fill envelope and the integrity of the fill has not been compromised by interim construction, environmental degradation and/or disturbance by the footing excavation.
13. Any excavation carried out in certified engineered fill must be reported to the geotechnical consultant who supervised the fill placement in order to document the locations of excavation and/or to supervise reinstatement of the excavated areas to engineered fill status. If construction on the engineered fill

does not commence within a period of 2 years from the date of certification, the condition of the engineered fill must be assessed for recertification.

14. Despite stringent control in the placement of the engineered fill, variations in soil type and density may occur in the engineered fill. Therefore, the strip footings and the upper section of the foundation walls constructed on the engineered fill may require continuous reinforcement with steel bars, depending on the uniformity of the soils in the engineered fill and the thickness of the engineered fill underlying the foundations. Should the footings and/or walls require reinforcement, the required number and size of reinforcing bars must be assessed by considering the uniformity as well as the thickness of the engineered fill beneath the foundations. In sewer construction, the engineered fill is considered to have the same structural proficiency as a natural inorganic soil.

6.3 Slab-On-Grade

The surface of the subgrade must be inspected and proof-rolled. Any topsoil or highly weathered soils must be subexcavated and replaced with inorganic fill, compacted to at least 98% of its maximum Standard Proctor dry density prior to placement of the granular base.

The slab should be constructed on a granular base, 20 cm thick, consisting of 20-mm Crusher-Run Limestone, or equivalent, compacted to 100% of its maximum Standard Proctor dry density.

A Modulus of Subgrade Reaction of 25 MPa/m is recommended for the design of the floor slab.

The ground around the building must be graded to direct water away from the structure to minimize the frost heave phenomenon generally associated with the disclosed soils.

6.4 Garages, Driveways, Sidewalks and Interlocking Stone Pavement

Due to the high frost susceptibility of the underlying soils, heaving of the pavement is expected to occur during the cold weather. The driveways at the entrances to the garage should be backfilled with non-frost-susceptible granular material, with a frost taper at a slope of 1 vertical:1 horizontal. The garage floor slab and interior garage foundation walls must be insulated with 50-mm Styrofoam, or equivalent.

Interlocking stone pavement and the sidewalks in areas which are sensitive to frost-induced ground movement, such as entrances, must be constructed on a free-draining, non-frost-susceptible granular material such as Granular ‘B’. It must extend to 1.2 m below the slab or pavement surface and be provided with positive drainage such as weeper subdrains connected to manholes or catch basins. Alternatively, the sidewalks and the interlocking stone pavement should be properly insulated with 50-mm Styrofoam, or equivalent, as approved by a geotechnical engineer.

The external grade must be designed to slope away from the structure.

6.5 Underground Services

The subgrade for the underground services should consist of properly compacted inorganic earth fill or natural sound soils. In areas consisting of topsoil or highly weathered soils, they should be subexcavated and replaced with bedding material compacted to at least 95% or + of its Standard Proctor compaction.

A Class ‘B’ bedding, consisting of compacted 20-mm Crusher-Run Limestone or Brampton Class ‘B’ bedding stone, is recommended for the construction of the underground services.

In order to prevent pipe floatation when the sewer trench is deluged with water, a soil cover with a thickness equal to the diameter of the pipe should be in place at all times after completion of the pipe installation.

Openings to subdrains and catch basins should be shielded with a fabric filter to prevent blockage by silting.

Since the silty clay has moderately high corrosivity to buried metal, the water main should be protected against corrosion. In determining the mode of protection, an electrical resistivity of 2500 ohm·cm should be used. This, however, must be confirmed by testing the soil along the water main alignment at the time of sewer construction.

6.6 Trench Backfilling

The on site inorganic soils are suitable for trench backfill. In the zone within 1.0 m below the pavement subgrade, the backfill should be compacted to at least 98% of its maximum Standard Proctor dry density with the moisture content 2% to 3% drier than the optimum. In the lower zone, a 95% or + Standard Proctor compaction is considered to be adequate; however, the material must be compacted on the wet side of the optimum.

In normal underground services construction practice, the problem areas of road settlement largely occur adjacent to manholes, catch basins, services crossings, foundation walls and columns. In areas which are inaccessible to a heavy compactor, sand backfill should be used. The interface of the native soils and the sand backfill will have to be flooded for a period of several days.

The narrow trenches should be cut at 1 vertical:2 or + horizontal so that the backfill can be effectively compacted. Otherwise, soil arching will prevent the achievement of proper compaction. The lift of each backfill layer should either be limited to a thickness of 20 cm, or the thickness should be determined by test strips.

One must be aware of the possible consequences during trench backfilling and exercise caution as described below:

- When construction is carried out in freezing winter weather, allowance should be made for these following conditions. Despite stringent backfill monitoring, frozen soil layers may inadvertently be mixed with the structural trench backfill. Should the in situ soil have a water content on the dry side of the optimum, it would be impossible to wet the soil due to the freezing condition, rendering difficulties in obtaining uniform and proper compaction. Furthermore, the freezing condition will prevent flooding of the backfill when it is required, such as in a narrow vertical trench section, or when the trench box is removed. The above will invariably cause backfill settlement that may become evident within 1 to several years, depending on the depth of the trench which has been backfilled.
- In areas where the underground services construction is carried out during winter months, prolonged exposure of the trench walls will result in frost heave within the soil mantle of the walls. This may result in some settlement as the

frost recedes, and repair costs will be incurred prior to final surfacing of the new pavement and the slab-on-grade construction.

- To backfill a deep trench, one must be aware that future settlement is to be expected, unless the side of the cut is flattened to at least 1 vertical: 1.5 + horizontal, and the lifts of the fill and its moisture content are stringently controlled; i.e., lifts should be no more than 20 cm (or less if the backfilling conditions dictate) and uniformly compacted to achieve at least 95% of the maximum Standard Proctor dry density, with the moisture content on the wet side of the optimum.
- It is often difficult to achieve uniform compaction of the backfill in the lower vertical section of a trench which is an open cut or is stabilized by a trench box, particularly in the sector close to the trench walls or the sides of the box. These sectors must be backfilled with sand. In a trench stabilized by a trench box, the void left after the removal of the box will be filled by the backfill. It is necessary to backfill this sector with sand, and the compacted backfill must be flooded for 1 day, prior to the placement of the backfill above this sector, i.e., in the upper sloped trench section. This measure is necessary in order to prevent consolidation of inadvertent voids and loose backfill which will compromise the compaction of the backfill in the upper section. In areas where groundwater movement is expected in the sand fill mantle, anti-seepage collars should be provided.

6.7 **Pavement Design**

Based on the borehole findings, the recommended pavement design is given in Table 4.

Table 4 - Pavement Design

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	50	HL-8
Granular Base	150	20-mm Crusher-Run Limestone
Granular Sub-base Parking Fire Route	 300 400	50-mm Crusher-Run Limestone

In preparation of the subgrade, the subgrade surface should be proof-rolled; any soft subgrade, organics and deleterious materials within 1.0 m below the underside of the granular sub-base should be subexcavated and replaced by properly compacted organic-free earth fill or granular material.

All the granular bases should be compacted to their maximum Standard Proctor dry density.

In the zone within 1.0 m below the pavement subgrade, the backfill should be compacted to at least 98% of its maximum Standard Proctor dry density, with the water content 2% to 3% drier than the optimum. In the lower zone, a 95% or + Standard Proctor compaction is considered adequate.

The road subgrade will suffer a strength regression if water is allowed to infiltrate prior to paving. The following measures should therefore be incorporated in the construction procedures and road design:

- If the road construction does not immediately follow the trench backfilling, the subgrade should be properly crowned and smooth-rolled to allow interim precipitation to be properly drained.
- Areas adjacent to the roads should be properly graded to prevent the ponding of large amounts of water during the interim construction period.
- Curb subdrains will be required. The subdrains should consist of filter-sleeved weepers to prevent blockage by silting.
- If the roads are to be constructed during the wet seasons and extensively soft subgrade occurs, the granular sub-base may require thickening. This can be assessed during construction.

6.8 Soil Parameters

The recommended soil parameters for the project design are given in Table 6.

Table 6 - Soil Parameters

Unit Weight and Bulk Factor	**Unit Weight (kN/m^3)**	**Estimated Bulk Factor**	
	Bulk	**Loose**	**Compacted**
Weathered Soil	20.5	1.20	0.98
Silty Clay	20.5	1.30	1.00
Silts	20.5	1.20	1.00
Lateral Earth Pressure Coefficients			
	Active K_a	**At Rest K_o**	**Passive K_p**
Silty Clay	0.39	0.56	2.56
Silts	0.33	0.50	3.00

6.9 Excavation

Excavations should be carried out in accordance with Ontario Regulation 213/91.

Excavations in excess of 1.2 m should be sloped at 1 vertical:1 horizontal for stability.

For excavation purposes, the types of soils are classified in Table 7.

Table 7 - Classification of Soils for Excavation

Material	Type
Silty Clay, weathered Soil and Silts above groundwater	3
Silts below groundwater	4

The groundwater yield from the silty clay, due to its low permeability, will be small, if any, and can be controlled by pumping from sumps. However, the yield from the silts will likely be moderate, if encountered; pumping from closely spaced sumps or, if necessary, a well-point dewatering system will be required.

Prospective contractors must be asked to assess the in situ subsurface conditions for soil cuts by digging test pits to at least 0.5 m below the intended bottom of excavation. These test pits should be allowed to remain open for a period of at least 4 hours to assess the trenching conditions.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of The Town of Newmarket, and for review by its designated agents, financial institutions, and government agencies Use of the report is subject to the conditions and limitations of the contractual agreement. The material in the report reflects the judgment of Frank Lee, P.Eng., and Bernard Lee, P.Eng., in light of the information available to it at the time of preparation. Any use which a Third Party makes of this report, and/or any reliance on decisions to be made based on it are the responsibility of such Third Parties. Soil Engineers Ltd. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

SOIL ENGINEERS LTD.

Frank Lee, P.Eng.

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FL/BL:dd

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PROVINCE OF ONTARIO

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LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

The abbreviations and terms commonly employed on the borehole logs and figures, and in the text of the report, are as follows:

SAMPLE TYPES

AS	Auger sample
CS	Chunk sample
DO	Drive open (split spoon)
DS	Denison type sample
FS	Foil sample
RC	Rock core (with size and percentage recovery)
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

PENETRATION RESISTANCE

Dynamic Cone Penetration Resistance:

A continuous profile showing the number of blows for each foot of penetration of a 2-inch diameter, 90° point cone driven by a 140-pound hammer falling 30 inches.

Plotted as '—•—'

Standard Penetration Resistance or 'N' Value:

The number of blows of a 140-pound hammer falling 30 inches required to advance a 2-inch O.D. drive open sampler one foot into undisturbed soil.

Plotted as '○'

WH	Sampler advanced by static weight
PH	Sampler advanced by hydraulic pressure
PM	Sampler advanced by manual pressure
NP	No penetration

SOIL DESCRIPTION

Cohesionless Soils:

'N' (blows/ft)	Relative Density
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

Cohesive Soils:

Undrained Shear Strength (ksf)	'N' (blows/ft)	Consistency
less than 0.25	0 to 2	very soft
0.25 to 0.50	2 to 4	soft
0.50 to 1.0	4 to 8	firm
1.0 to 2.0	8 to 16	stiff
2.0 to 4.0	16 to 32	very stiff
over 4.0	over 32	hard

Method of Determination of Undrained Shear Strength of Cohesive Soils:

x 0.0 Field vane test in borehole; the number denotes the sensitivity to remoulding

△ Laboratory vane test

☐ Compression test in laboratory

For a saturated cohesive soil, the undrained shear strength is taken as one half of the undrained compressive strength

METRIC CONVERSION FACTORS

1 ft = 0.3048 metres
1lb = 0.454 kg

1 inch = 25.4 mm
1ksf = 47.88 kPa

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JOB NO.: 1708-S019

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed Existing Residential Property

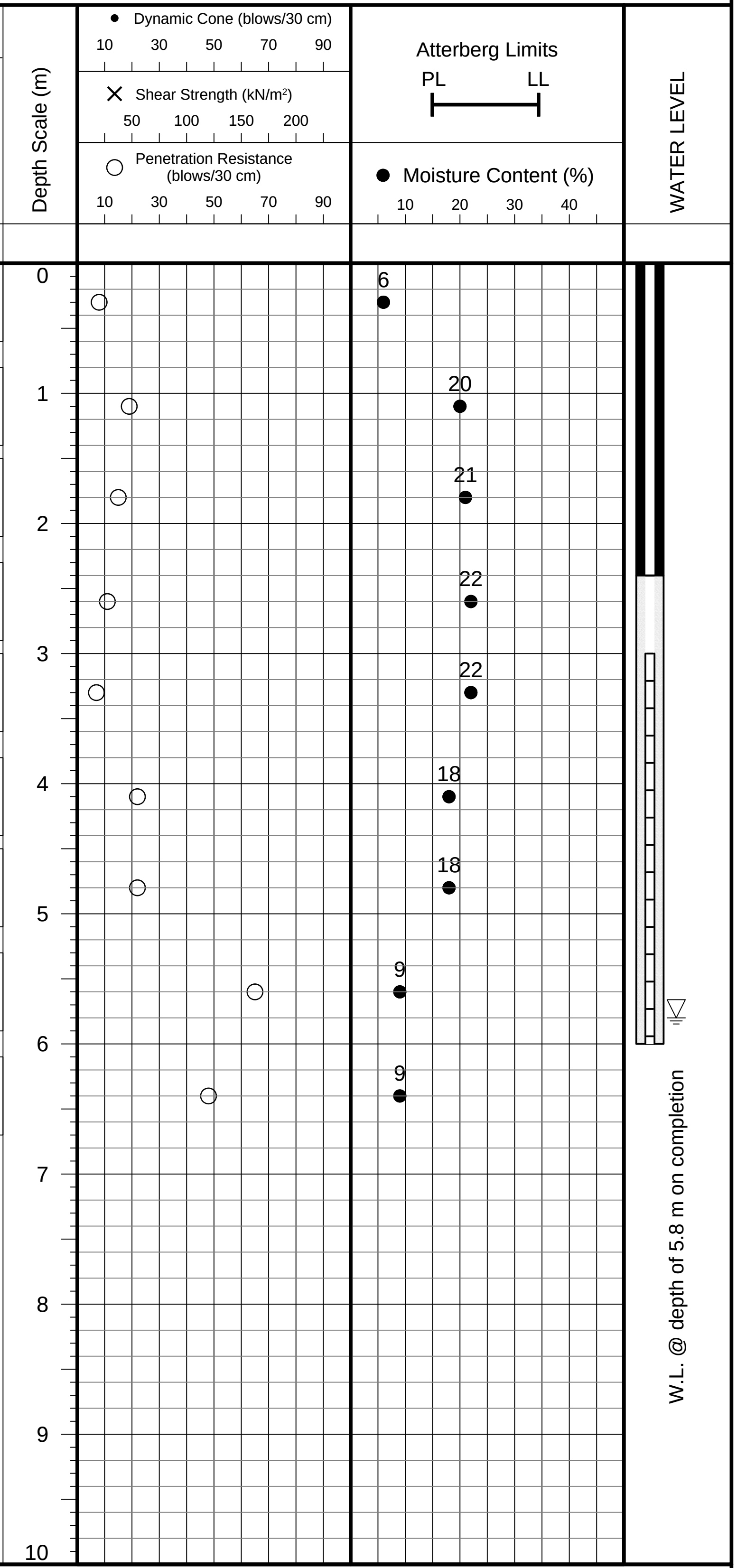
METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 16780 Yonge Street, Town of Newmarket

DRILLING DATE: September 1, 2017

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
0.0	Ground Surface			
0.0 0.3	**25 cm TOPSOIL**	1	DO	8
	Loose to compact weathered			
		2	DO	19
		3	DO	15
	SILT	4	DO	11
		5	DO	7
	brown grey			
		6	DO	22
		7	DO	22
5.3	Grey, dense to very dense **SANDY SILT**	8	DO	65
		9	DO	48
6.7	**END OF BOREHOLE**			

Installed 50 mm Ø monitoring well to 6.0 m
completed with 3.0 m screen
Sand backfill from 2.4 m to 6.0 m
Bentonite seal from 0.0 m to 2.4 m
Provided with a protective steel casing



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JOB NO.: 1708-S019

LOG OF BOREHOLE NO.: 6

FIGURE NO.: 2

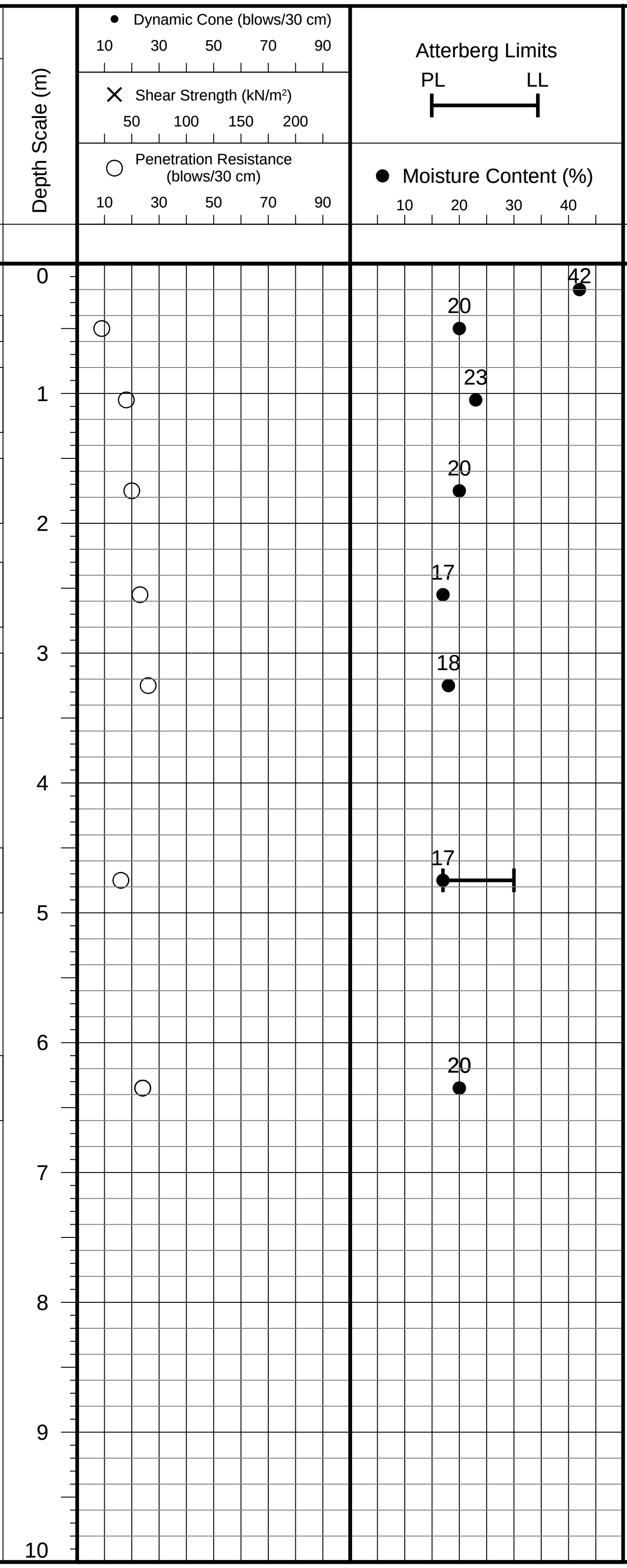
PROJECT DESCRIPTION: Proposed Existing Residential Property

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 16780 Yonge Street, Town of Newmarket

DRILLING DATE: September 1, 2017

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	WATER LEVEL
0.0	Ground Surface				
0.0	**38 cm TOPSOIL**	1A	DO		Dry on completion
	Brown, loose to compact weathered **SILT**	1B	DO	9	
		2	DO	18	
		3	DO	20	
		4	DO	23	
		5	DO	26	
4.0	Grey, stiff to very stiff **SILTY CLAY** occ. wet sand and silt seams and layers	6	DO	16	
		7	DO	24	
6.6	**END OF BOREHOLE**				



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JOB NO.: 1708-S019

LOG OF BOREHOLE NO.: 7

FIGURE NO.: 3

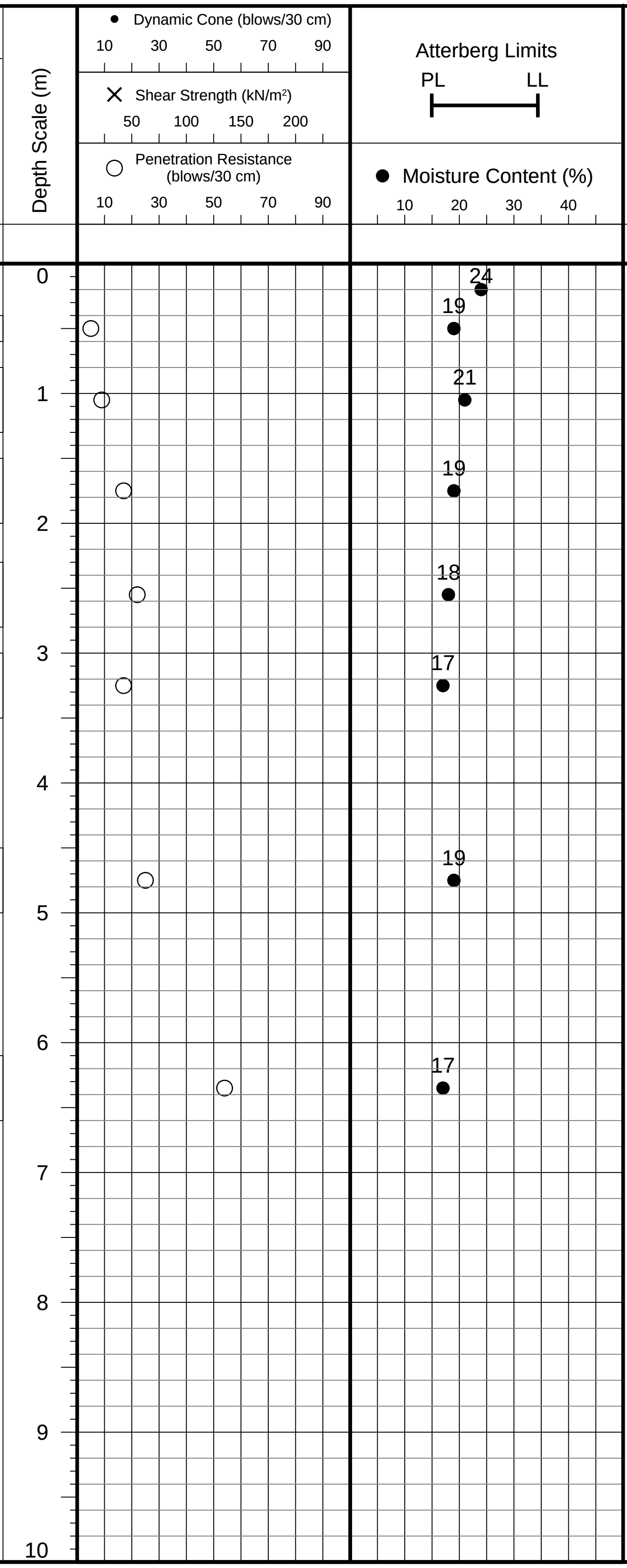
PROJECT DESCRIPTION: Proposed Existing Residential Property

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 16780 Yonge Street, Town of Newmarket

DRILLING DATE: September 1, 2017

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value	WATER LEVEL
0.0	Ground Surface				
0.0	**38 cm TOPSOIL**	1A	DO		
		1B	DO	5	
	Brown, loose to very dense	2	DO	9	
	weathered	3	DO	17	
		4	DO	22	
	SILT	5	DO	17	Dry on completion
		6	DO	25	
		7	DO	54	
6.6	**END OF BOREHOLE**				



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JOB NO.: 1708-S019

LOG OF BOREHOLE NO.: 8

FIGURE NO.: 4

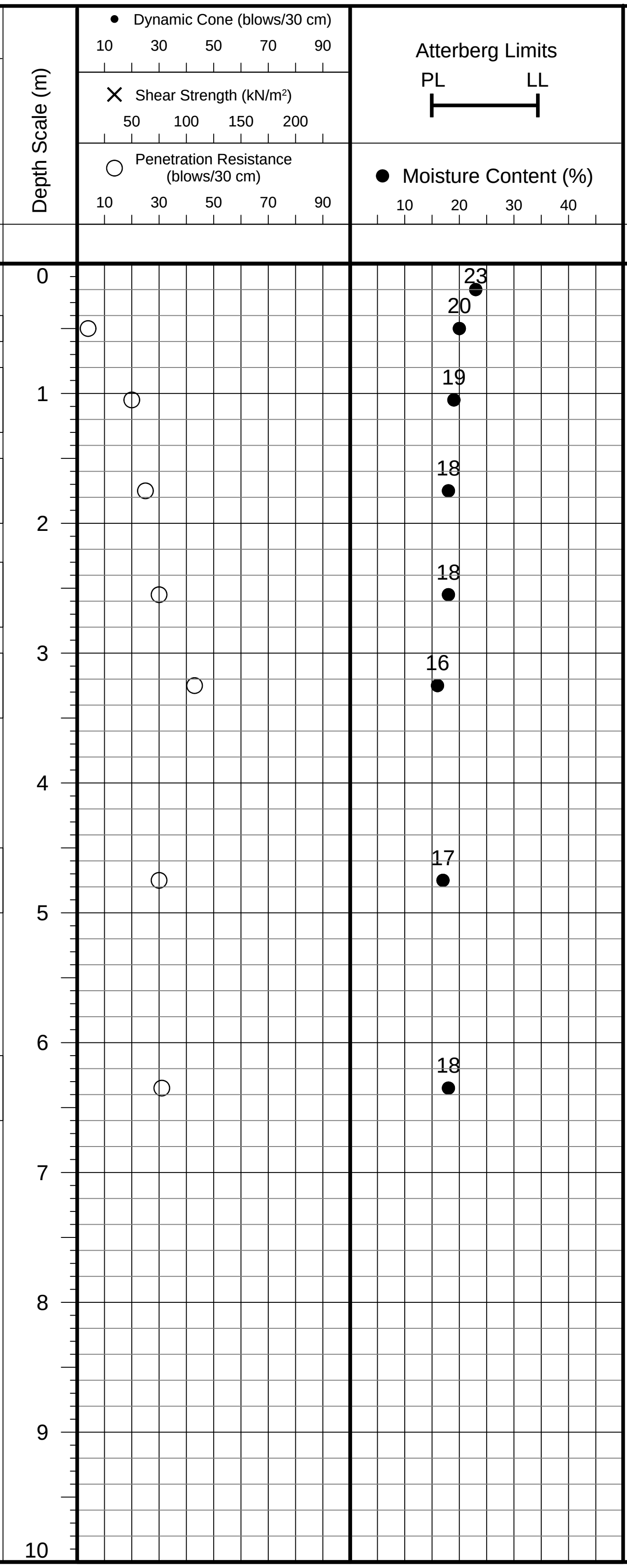
PROJECT DESCRIPTION: Proposed Existing Residential Property

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 16780 Yonge Street, Town of Newmarket

DRILLING DATE: September 1, 2017

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	WATER LEVEL
0.0	Ground Surface				
0.0	**36 cm TOPSOIL**	1A	DO		Dry on completion
	Brown, loose to dense weathered	1B	DO	4	
	SILT	2	DO	20	
		3	DO	25	
		4	DO	30	
		5	DO	43	
4.0	Grey, very stiff **SILTY CLAY** occ. wet sand and silt seams and layers	6	DO	30	
		7	DO	31	
6.6	**END OF BOREHOLE**				



JOB NO.: 1708-S019

LOG OF BOREHOLE NO.: 9

FIGURE NO.: 5

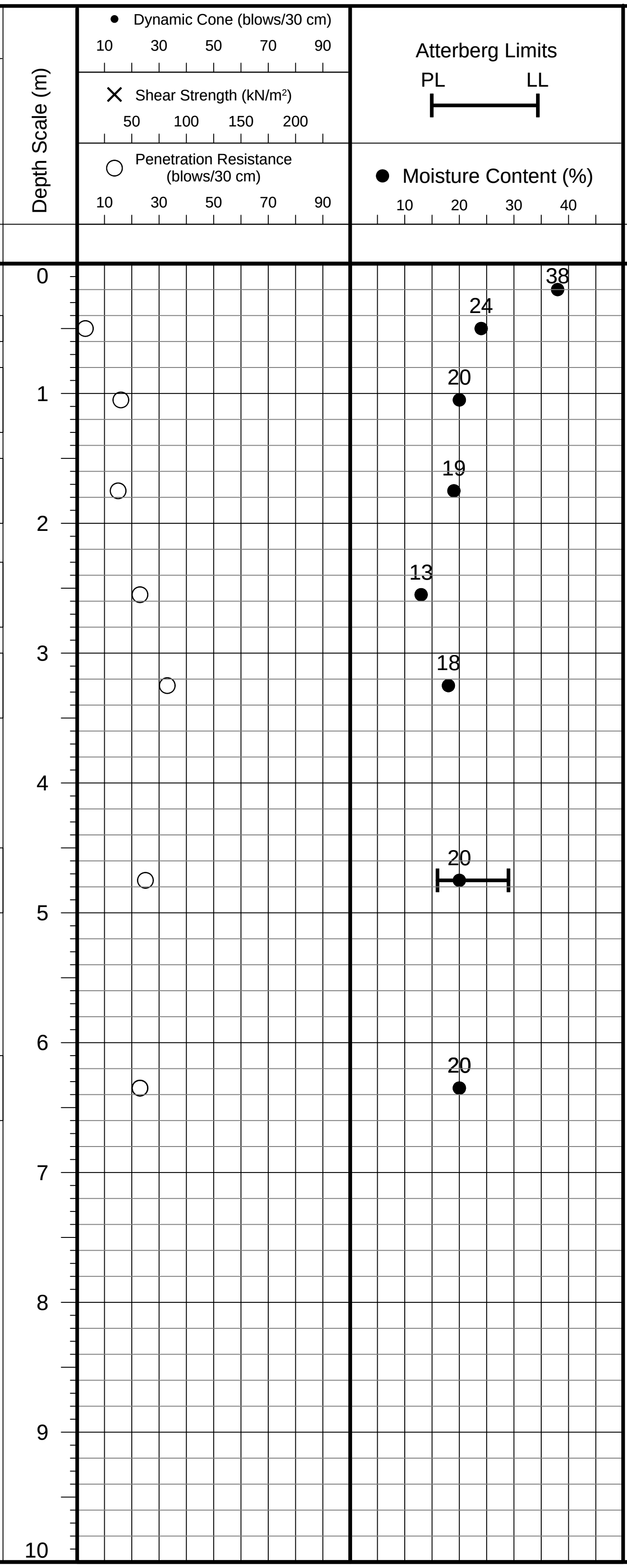
PROJECT DESCRIPTION: Proposed Existing Residential Property

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 16780 Yonge Street, Town of Newmarket

DRILLING DATE: September 1, 2017

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
0.0	Ground Surface			
0.0	**43 cm TOPSOIL**	1A	DO	
	Loose to dense weathered	1B	DO	3
	SILT	2	DO	16
		3	DO	15
		4	DO	23
		5	DO	33
4.0	Grey, very stiff **SILTY CLAY** occ. wet sand and silt seams and layers	6	DO	25
		7	DO	23
6.6	**END OF BOREHOLE**			



WATER LEVEL

W.L. @ depth of 4.3 m on completion

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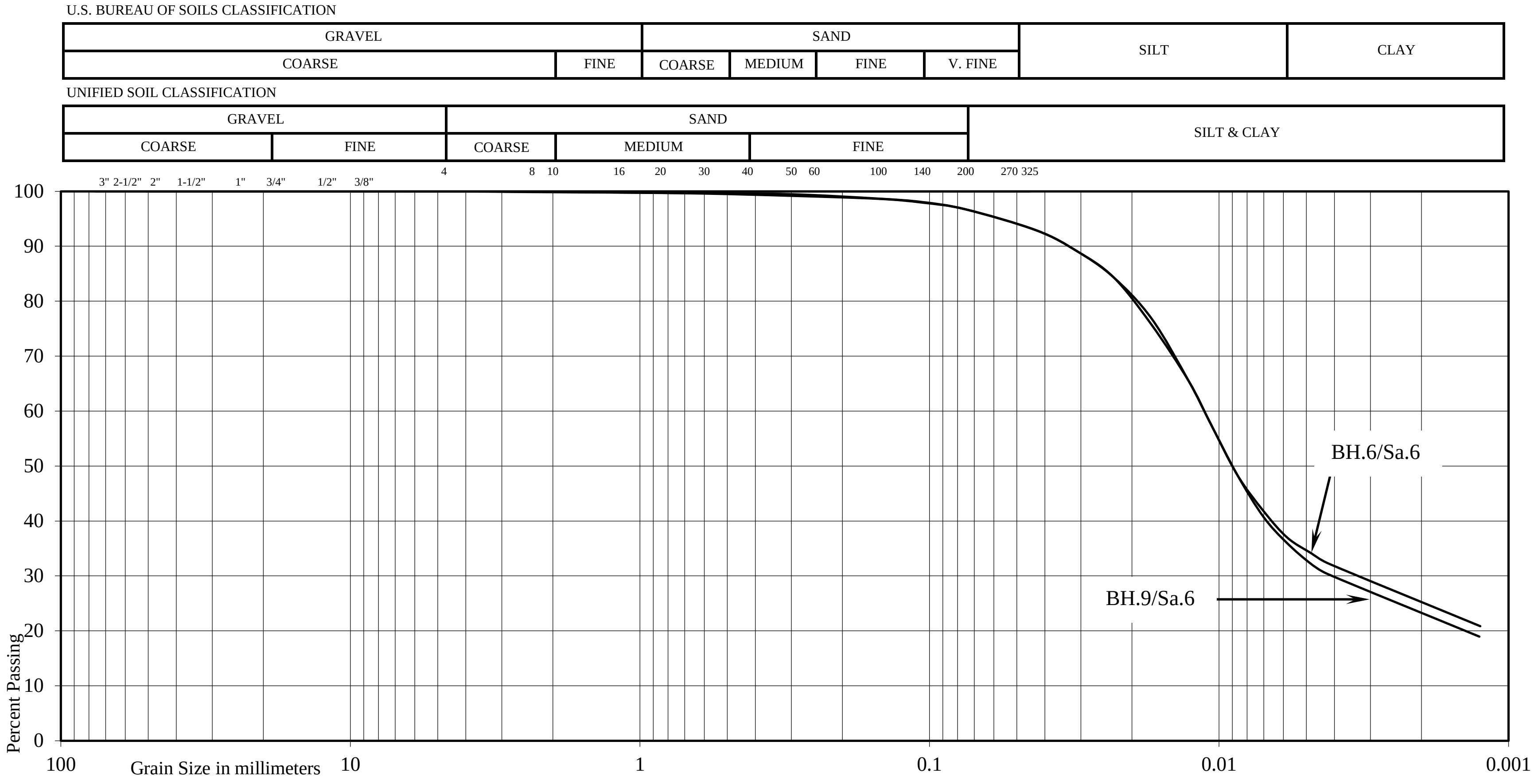
GRAIN SIZE DISTRIBUTION

Reference No: 1708-S019



Project: Existing Residential Property

Location: 16780 Yonge Street, Town of Newmarket

Borehole No:	6	9
Sample No:	6	6
Depth (m):	4.7	4.7
Elevation (m):	-	-

BH./Sa.	6/6	9/6
Liquid Limit (%) =	30	29
Plastic Limit (%) =	17	16
Plasticity Index (%) =	13	13
Moisture Content (%) =	17	20
Estimated Permeability (cm./sec.) =	10^{-7}	10^{-7}

Classification of Sample [& Group Symbol]: SILTY CLAY, a trace of fine sand

Figure: 6

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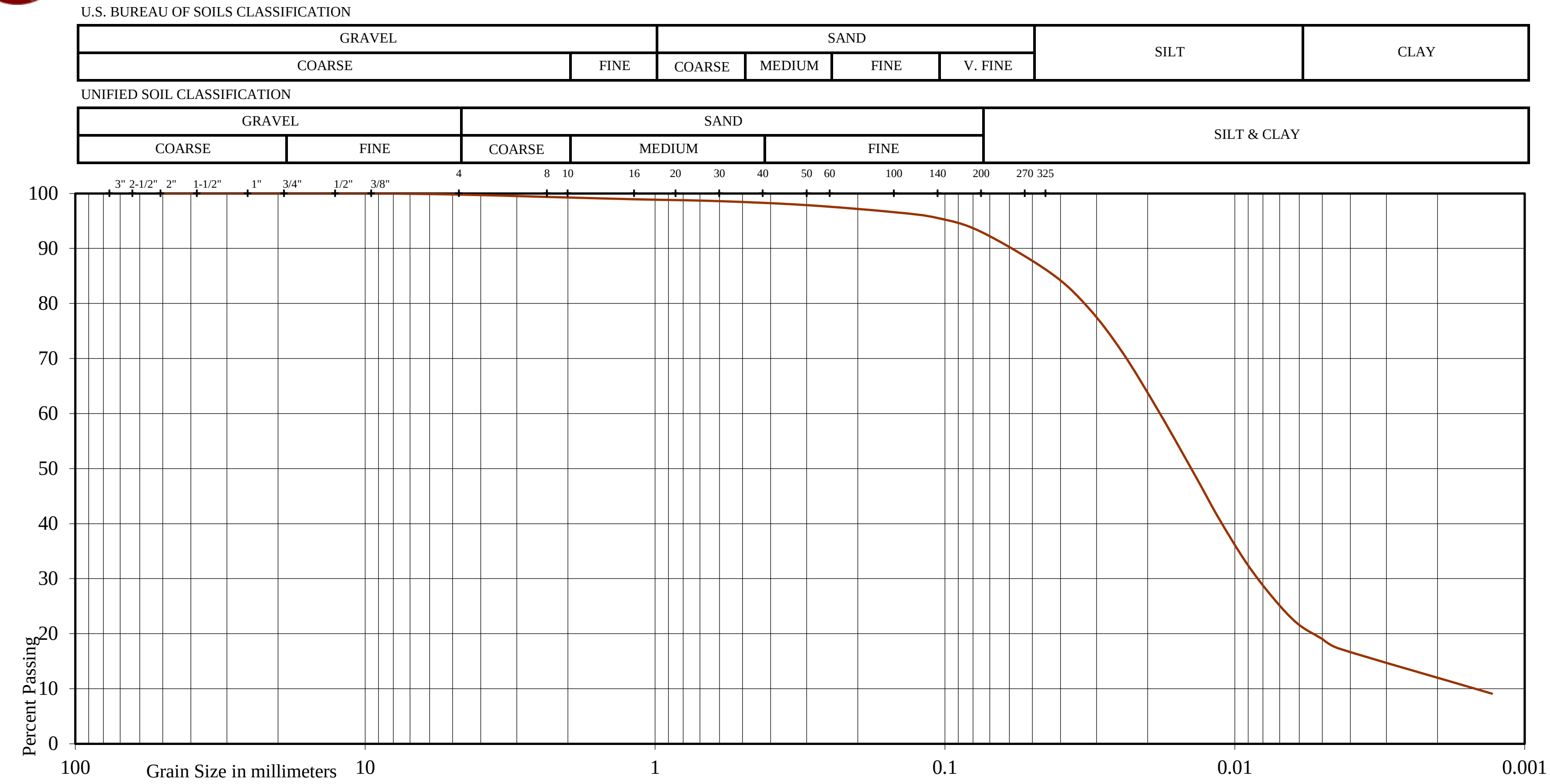
GRAIN SIZE DISTRIBUTION

Reference No: 1708-S019



Project:	Existing Residential Property
Location:	16780 Yonge Street, Town of Newmarket

Borehole No:	1
Sample No:	6
Depth (m):	4.2
Elevation (m):	-

Liquid Limit (%) =	-
Plastic Limit (%) =	-
Plasticity Index (%) =	-
Moisture Content (%) =	18
Estimated Permeability (cm./sec.) =	10^{-6}

Classification of Sample [& Group Symbol]: SILT, some clay, a trace of fine sand

Figure: 7

ENV BH/MW

ENV BH/MW (at approximate location

GEO BH/MW

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BOREHOLE LOCATION PLAN

DESIGNED	CHECKED	DWG NO. 1	REV
SCALE NTS	REF. NO. 1708-S019	DATE September 2017	

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SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

JOB NO.: 1708-S019
REPORT DATE: October 2017
PROJECT DESCRIPTION: Proposed Existing Residential Property
PROJECT LOCATION: 16780 Yonge Street, Town of Newmarket

LEGEND

▤ TOPSOIL ▥ SANDY SILT ▥ SILT ▨ SILTY CLAY

▽ WATER LEVEL (END OF DRILLING)