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A REPORT TO
WYCLIFFE HOMES

A GEOTECHNICAL INVESTIGATION FOR PROPOSED RESIDENTIAL DEVELOPMENT

3171 LAKESHORE ROAD WEST

TOWN OF OAKVILLE

REFERENCE NO. 1704-S067

MAY 2017

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1.0 INTRODUCTION

In accordance with the authorization dated April 13, 2017, from Mr. Gary Bensky of Wycliffe Homes, a geotechnical investigation was carried out at the parcel of property located at 3171 Lakeshore Road West, in the Town of Oakville.

The purpose of the investigation was to reveal the subsurface conditions and determine the engineering properties of the disclosed soils for the design and construction of a proposed Residential Development.

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

2.0 SITE AND PROJECT DESCRIPTION

The Town of Oakville is situated on Iroquois Lake plain where a drift overburden overlies a shale bedrock which occurs at a relatively shallow depth. The drift has been partly eroded and, in places, filled with lacustrine clay, silt, and sand.

The subject property, approximately 1.2 hectares in area, is located at 3171 Lakeshore Road West in the Town of Oakville. It was occupied by a garden centre.

A preliminary site plan of the proposed development indicates that the subject property will be developed into 27 residential lots, accessible by Lakeshore Road West and the proposed extension of Victoria Street, with municipal services meeting urban standards.

3.0 FIELD WORK

The field work, consisting of four (4) boreholes, was performed on April 25, 2017 at the locations shown on the Borehole Location Plan, Drawing No. 1. The boreholes extended to depths ranging from 4.7 to 6.2 m from the prevailing ground surface where refusal to augering on bedrock was encountered.

The boreholes were advanced at intervals to the sampling depths by a track-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 elevation at each of the borehole locations was determined using hand-held Global Navigation Satellite System survey equipment (Trimble Geoexplorer 6000), having an accuracy of 10 cm.

4.0 SUBSURFACE CONDITIONS

Detailed descriptions of the encountered subsurface conditions are presented on the Borehole Logs, comprising Figures 1 to 4, inclusive. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing No. 2. The engineering properties of the disclosed soils are discussed herein.

The boreholes revealed that beneath a veneer of topsoil and an earth fill, the site is generally underlain by strata of silt and silty clay, overlying the shale bedrock.

4.1 **Topsoil** (Boreholes 1 and 2)

Topsoil was encountered at the ground surface of the landscaped areas. At Boreholes 1 and 2, the topsoil is 10 cm and 30 cm in thickness. The topsoil is dark brown in colour and permeated with roots and humus. These materials are unstable and compressible under loads; therefore, the topsoil is considered to be void of engineering value. It can be used for general landscaping purpose only.

Due to the humus content, the topsoil will generate an offensive odour under anaerobic conditions and may produce volatile gases; therefore, it must not be buried within the building envelope, or deeper than 1.2 m below the finished grade, as it may have an adverse impact on the environmental well-being of the development.

4.2 **Earth Fill** (All Boreholes)

A layer of earth fill, extending to depths ranging from 1.0 to 2.3 m from grade, was encountered in the boreholes. It consists of sandy silt or silty sand, with occasional rootlet and topsoil inclusions.

The obtained ‘N’ values ranged from 3 to 22 blows per 30 cm penetration, showing that the earth fill was generally loose, with non-uniform compaction. The natural water content of the earth fill samples range from 9% to 19%, indicating moist to very moist conditions.

In using the earth fill for structural backfill, it must be subexcavated, inspected, sorted free of any serious topsoil inclusions, or other deleterious materials, and properly compacted in layers.

<u>One must be aware that the samples retrieved from boreholes 10 cm in diameter may not be truly representative of the geotechnical and environmental quality of the fill, and do not indicate whether the topsoil beneath the earth fill was completely stripped. This should be further assessed by laboratory testing and/or test pits.</u>

4.3 **Silt** (Boreholes 1, 2 and 4)

The silt deposit was generally encountered below the earth fill at 1.0 to 2.3 m from grade. Sample examinations show that the deposit is slightly cohesive, in a very moist condition. The natur6al water content values of the soil samples were determined at 17% to 21%.

The obtained ‘N’ values ranged from 6 to 26 blows, with a median of 16 per 30 cm of penetration, indicating a relative density of loose to compact, being generally compact.

A grain size analysis was performed on 1 representative sample of the silt and the result is plotted on Figure 5.

According to the above findings, the following engineering properties are deduced:

- High to high frost susceptibility and soil adfreezing potential.
- High water erodibility, susceptible to migration of soil particles through small openings under seepage pressure.
- Relatively low permeability, with an estimated coefficient of permeability of 10^{-6} cm/sec, an average percolation rate of 60 min/cm and runoff coefficients of:

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

- A frictional soil, its shear strength is dependent on its internal friction angle and soil density. Its shear strength 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 reduction of shear strength.
- In excavation, the wet silt will slough, run with seepage and boil with a piezometric head of about 0.4 m.
- A poor pavement-supportive material, with an estimated California Bearing Ratio (CBR) value of 3%.
- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 3000 ohm·cm.

4.4 **Silty Clay** (All Boreholes)

The silty clay was contacted below the silt or earth fill at depths of 1.5 to 3.3 m from grade. It is laminated with sand seams with shale fragments at the lower depth.

The consistency of the silty clay is stiff to hard, being generally very stiff, as confirmed by the obtained ‘N’ values between 11 and 32 blows, with a median of 16 blows per 30 cm of penetration.

The Atterberg Limits of 1 representative sample and the natural water content of all the clay samples were determined. The results are plotted on the Borehole Logs and summarized below:

Liquid Limit	25%
Plastic Limit	15%
Natural Water Content	5% to 21% (median 15%)

The above results show that the clay is cohesive material with low plasticity. The natural water content generally lies below and slightly above the plastic limits, confirming the consistency of the clay deposit as determined by the ‘N’ values.

A grain size analysis was performed on 1 representative sample of the silty clay; the result is plotted on Figure 6.

According to the above findings, the following engineering properties are deduced:

- Highly frost susceptible and low water erodible.
- Virtually impervious, with an estimated coefficient of permeability of 10^{-7} cm/sec, an average percolation rate of 80 min/cm, and runoff coefficients of:

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

- A cohesive soil, its shear strength is derived from consistency and augmented by the internal friction of the silt. Its shear strength is moisture dependent and, due to the dilatancy of the silt, the overall shear strength of the silty clay 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 of shear strength.
- It will generally be stable in a relatively steep cut; however, prolonged exposure will allow the sand seams to become saturated which may lead to localized sloughing.
- A poor pavement-supportive material, with an estimated CBR of 3%.
- Moderately high corrosivity to buried metal, with an estimated electrical resistivity of 3000 ohm·cm.

4.5 **Shale** (All Boreholes)

Weathered shale was encountered beneath the silty clay in all the boreholes. It is reddish-brown in colour indicating a Queenston formation. The quality of the shale bedrock, is not proven by rock coring. The shale is susceptible to disintegration and swelling upon exposure to air and water, with subsequent reversion to a clay soil, but the laminated limy and sandy layers would remain as rock slabs. The shale within the borehole depth can be penetrated by power-augering with some difficulty in grinding through the hard layers.

The shale has a low permeability and occasional pockets of groundwater trapped in its fissures have been encountered. This water may be under a moderate subterranean artesian pressure but, upon release through excavation, the water is likely to drain readily with a limited yield.

The weathered rock can be excavated with considerable effort by a heavy-duty backhoe equipped with a rock-ripper; however, excavation will become progressively more difficult with depth into the sound shale. Efficient removal of the sound shale may require the aid of blasting or pneumatic hammering.

The excavated spoil will contain a large amount of hard limy and sandy rock slabs, rendering it virtually impossible to obtain uniform compaction. Therefore, unless the spoil is sorted, it is considered unsuitable for engineering applications.
In sound shale excavation, slight lateral displacement of the excavation walls is often experienced. This is due to the release of residual stress stored in the bedrock mantle and the swelling characteristic of the rock

4.6 **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 +**
Earth Fill	9 to 19	11	7 to 15
Silty Clay	5 to 21	17	13 to 22
Silt	17 to 21	12	8 to 17

Based on the above findings, the in-situ material is mostly on the wet side that it will require aeration before compaction for 95% or + Standard Proctor compaction. The aeration can be conducted by spreading thinly on the ground during the dry and warm weather.

The on site material should be compacted using a heavy-weight kneading-type roller. The sand and silt can be compacted by a smooth drum 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.

When compacting the silty clay on the dry side of the optimum, the compactive energy will frequently bridge over the chunks in the soil and be transmitted laterally into the soil mantle. Therefore, the lifts of these soils must be limited to 20 cm or less (before compaction). 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 road 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 silt and sands 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 road 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 foundations 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.

As noted, the shale is susceptible to disintegration and will revert to a clay soil. The shale spoil which has been exposed to weathering may be selected for use as structural fill. To achieve this, the shale must be excavated by a rock-ripper to break up the limy shale and sandstone slabs, and piled thinly on the ground for optimum exposure to weathering.

5.0 GROUNDWATER CONDITIONS

The boreholes were checked for the presence of groundwater or the occurrence of cave-in upon completion of the field work. The findings are summarized in Table 2:

Table 2 - Groundwater Levels

Borehole No.	**Ground El. (m)**	**Measured Groundwater/Cave-In* Level Upon Completion**	
		Depth (m)	**El. (m)**
1	84.8	2.0	82.8
2	84.4	2.0	82.4
3	85.9	2.3	83.6
4	86.4	1.7*	84.7*

*Cave-in level (In wet silts, the level generally represents the groundwater regime at the borehole location).

Upon completion, the groundwater or cave-in level was recorded between El. 82.4 m and 84.7 m. This may represent the percolated water from the ground surface, which is perched in the earth fill or silt deposit. The groundwater level may fluctuate with seasons.

In excavations below the saturated levels, the groundwater yield may be appreciable and likely persistent. The quantity will slow down after sometime. The groundwater can be collected in sumps and removed by conventional pumping.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation has disclosed that beneath a veneer of topsoil and an earth fill, the site is generally underlain by strata of silt and silty clay to a depth of 2.7 to 4.8 m. Weathered shale was encountered below the silt and clay deposits.

Upon completion, the groundwater or cave-in level was recorded between El. 82.4 m and 84.7 m. This may represent the percolated water from the ground surface, which is perched in the earth fill or silt deposit. The groundwater level may fluctuate with seasons. In excavations below the saturated levels, the groundwater yield may be appreciable and likely persistent. The quantity will slow down after sometime. The groundwater can be collected in sumps and removed by conventional pumping.

The subject property will be developed into 27 residential lots, accessible by the Lakeshore Road West and the proposed extension of Victoria Street, with municipal services and roadways meeting urban standards.

The geotechnical findings which warrant special consideration are presented below:

1. The revealed topsoil thickness is 10 cm and 30 cm in some of the boreholes. The topsoil thickness may vary randomly. Thicker topsoil layers can occur in other areas, especially in tree-covered areas.
2. The topsoil is void of engineering value and should be stripped and removed for the project construction. The topsoil must not be buried within the building envelope or deeper than 1.2 m below the exterior finished grade of the development. It should only be used for landscaping and landscape contouring purposes.
3. The existing earth fill is not suitable to support any structure sensitive to movement. It must be subexcavated and sorted free of topsoil inclusions or

deleterious materials before it is reused as engineered fill or structural backfill.

4. The sound natural soils below the earth fill is suitable for normal spread and strip footing construction for the proposed development. The footings must be designed in accordance with the recommended bearing pressures in Section 6.1 and the footing subgrade must be inspected by a geotechnical engineer to ensure that its condition is compatible with the design of the foundations.
5. Where the site will be regraded with earth fill, it is more economical to place an engineered fill for normal footing, sewer and pavement construction.
6. A Class ‘B’ bedding, consisting of compacted 20-mm Crusher-Run Limestone, or equivalent, is generally recommended for the construction of the underground services. Where saturated soils are present or extensive dewatering is required, a Class ‘A’ bedding will likely be required, and the pipe joints should be leak proof or wrapped with a waterproof membrane.
7. All excavation should be carried out in accordance with Ontario Regulation 213/91.

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**

The proposed development will consist of residential dwellings with basement. Based on the borehole findings, the dwellings can be constructed on conventional footings founded on the sound natural soils. The recommended soil bearing pressures for use in the design of normal strip and spread footings, together with the corresponding founding levels, are presented in Table 3.

Table 3 - Founding Levels

Borehole No.	**Recommended Maximum Allowable Soil Pressure (SLS)/ Factored Ultimate Soil Bearing Pressure (ULS) and Corresponding Founding Level**					
	75 kPa (SLS)		**150 kPa (SLS)**		**800 kPa (SLS)**	
	120 kPa (ULS)		**240 kPa (ULS)**		**1200 kPa (ULS)**	
	Depth (m)	**El. (m)**	**Depth (m)**	**El. (m)**	**Depth (m)**	**El. (m)**
1	1.2 or +	83.6 or -	2.5 or +	82.3 or -	4.6 or +	80.2 or -
2	2.5 or +	81.9 or -	3.4 or +	81.0 or -	3.8 or +	80.6 or -
3	-		1.7 or +	84.2 or -	2.7 or -	83.2 or -
4	-	-	1.7 or +	84.7 or -	4.9 or -	81.5 or -

Where extended footings and/or cut and fill is required for site grading, it is generally more economical to place engineered fill for normal footing, sewer and pavement construction. Recommended soil bearing pressures of 100 kPa (SLS) and 150 kPa (ULS) can be used for the design of the normal spread and strip foundations founded on engineered fill. The requirements for engineered fill construction are discussed in Section 6.2.

The recommended soil pressures (SLS) incorporate a safety factor of 3. The total and differential settlements of the footings are estimated to be 25 mm and 15 mm, respectively. If any part of the structure is founded on shale bedrock, the entire structure should be subexcavated onto the shale to prevent any cracks on the foundation due to abrupt differential settlement between the different bearing materials.

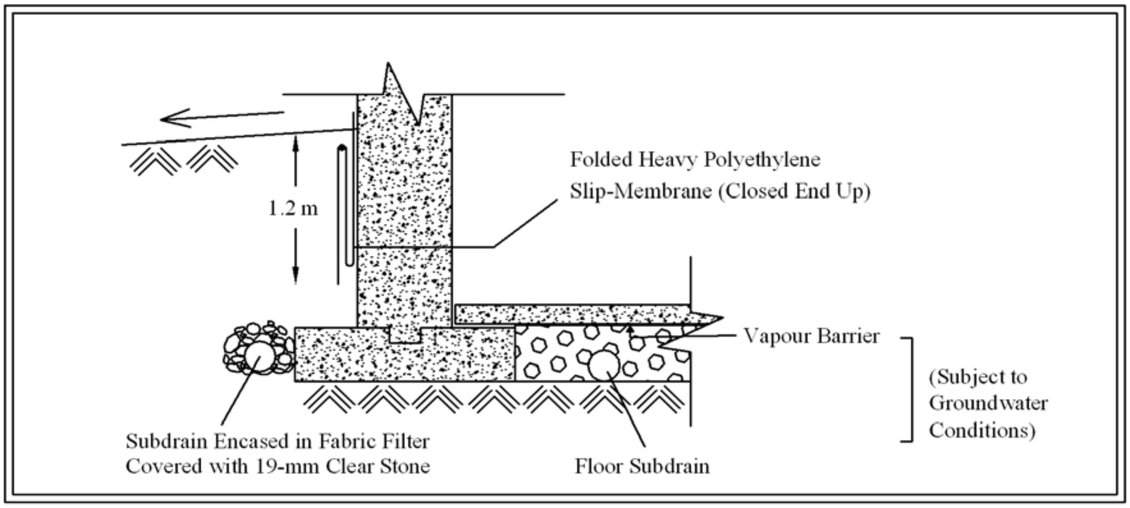
One must be aware the recommended Soil Bearing Pressures and the corresponding founding depths are given as a guide for foundation design and must be confirmed by subgrade inspection performed by a geotechnical engineer or a geotechnical technician under the supervision of a geotechnical engineer, to ensure that the revealed conditions are compatible with the foundation design requirements.

Footings exposed to weathering, or in unheated areas, should have at least 1.2 m of earth cover for protection against frost action.

The building foundation must meet the requirements specified in the latest Ontario Building Code. As a guide, the structure should be designed to resist an earthquake force using Site Classification ‘D’ (stiff soil). If the foundation is founded on shale, the structure should be designed to resist an earthquake force using Site Classification ‘C’ (dense soil).

The on site material have high soil-adfreezing potential. In order to alleviate the risk of frost damage, the foundation walls must be constructed of concrete and either the backfill must consist of non-frost-susceptible granular material, or the foundation walls must be shielded with a polyethylene slip-membrane between the concrete wall and the backfill. The recommended measures are schematically illustrated in Diagram 1.

Diagram 1 - Frost Protection Measures (Foundations)



Perimeter subdrains and dampproofing of the foundation walls will be required for the project construction. If wet silt or sand is encountered at the basement subgrade, under-floor subdrains and vapour barrier will be required. All subdrains must be encased in a fabric filter to protect them against blockage by silting.

6.2 **Engineered Fill**

Where earth fill is required to raise the site, or where extended footings are necessary, it is generally more economical to place engineered fill for normal footing, sewer and road construction.

The engineering requirements for a certifiable fill for road construction, municipal services, and footings designed with a Maximum Allowable Soil Pressure (SLS) of 100 kPa and a Factored Ultimate Soil Bearing Pressure (ULS) of 150 kPa are presented below:

1. All of the topsoil must be removed, and the subgrade must be inspected and proof-rolled prior to any fill placement. The existing earth fill and weathered soils must be subexcavated, inspected, aerated and properly compacted in layers.
2. The in situ organic-free soils can 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 lot grade and/or road subgrade. The soil moisture must be properly controlled. Aeration of the wet soils will be required prior to compaction.
3. If the building 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.

4. If imported fill is to be used, it should be inorganic soils, free of deleterious or any 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 hauling to the site.
5. 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.
6. The engineered fill must extend over the entire graded area; the engineered fill envelope and the 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, depending on the thickness of the fill, 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 (estimated to be 15± mm) between the natural soils and engineered fill.
7. 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.
8. 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.
9. Where the fill is to be placed on a bank steeper than 1V:3H, 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. The fill operation must be inspected on a full-time basis by a technician under the direction of a geotechnical engineer.

11. The footings and underground services subgrade must be inspected by the geotechnical consulting firm that inspected 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.
12. Any excavation carried out in certified engineered fill must be reported to the geotechnical consultant who inspected the fill placement in order to document the locations of excavation and/or to inspect 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 re-certification.
13. Despite stringent control in the placement of 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 will 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 **Underground Services**

The subgrade for the underground services should consist of natural soils or engineered fill. In areas where the subgrade consists of earth fill, it should be subexcavated and replaced with properly compacted inorganic soil and/or bedding material compacted to at least 95% or + of their Standard Proctor compaction.

Where the sewers are to be constructed using the open-cut method, the construction must be carried out in accordance with Ontario Regulation 213/91. In areas where a vertical cut is necessary, the use of a trench box is considered to be appropriate. In the design of the trench box and/or shoring structure, the lateral earth pressure coefficients presented in Table 5, Section 6.7, can be used.

A Class ‘B’ bedding is recommended for construction of the underground services. The bedding material should consist of compacted 20-mm Crusher-Run Limestone, or equivalent, as approved by a geotechnical engineer. Where wet silt or sand is encountered at the trench subgrade, a Class ‘A’ bedding should be used.

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.

The underground service trenches will consist of clay or silt soils of high to moderately high corrosivity. The underground services should be protected against soil corrosion. For estimation of anode weight requirements, the estimated electrical resistivity of 3000 ohm·cm can be used. This, however, should be confirmed by testing the soil along the pipe alignment at the time of construction.

6.4 Backfilling in Trenches and Excavated Areas

The backfill in service trenches should be compacted to at least 95% of its maximum Standard Proctor dry density. In the zone within 1.0 m below the road subgrade, the material should be compacted with the water content 2% to 3% drier than the optimum; and the compaction should be increased to 98% of the respective maximum Standard Proctor dry density to provide the required stiffness for pavement construction.

The on-site inorganic soils are generally suitable for use as trench backfill; however, wet soils must be aerated by spreading it thinly on the ground for drying prior to structural compaction.

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

Narrow trenches for services crossings should be cut at 1V:2H, so that the backfill in the trenches can be effectively compacted. Otherwise, soil arching in the trenches 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 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 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.
- 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 1V:1.5+H, 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.

6.5 Garages, Driveways and Landscaping

Due to high frost susceptibility of the underlying soil, heaving of the pavement is expected to occur during the cold weather.

The sidewalk and driveways at the entrances to the garages must be backfilled with non-frost-susceptible granular material, with a frost taper at a slope of 1V:1H.

The slab-on-grade in open areas should be designed to tolerate frost heave, and the grading around the slab-on-grade must be such that it directs runoff away from the surface.

Interlocking stone pavement and slab-on-grade to be constructed in areas susceptible to ground movement must be constructed on a free-draining granular base at least 1.0 m thick, with proper drainage, which will prevent water from ponding in the granular base.

6.6 Pavement Design

The recommended pavement design for the local residential road is presented in Table 4.

Table 4 - Pavement Design

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	60	HL-8
Granular Base	150	OPSS Granular ‘A’ or 20 mm Crusher-Run Limestone
Granular Sub-Base	350	OPSS Granular ‘B’ or 50 mm Crusher-Run Limestone

In preparation of the pavement subgrade, the topsoil must be removed and the areas should be proof-rolled. Any soft spots should be subexcavated, and replaced by properly compacted inorganic earth fill. New fill should be free of organic or deleterious material, compacted to 95% or + of its 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.

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

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

- If the pavement 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.
- Lot areas adjacent to the roads should be properly graded to prevent the ponding of large amounts of water during the interim construction period.

- If the roads are to be constructed during the wet seasons and extremely soft subgrade occurs, the granular sub-base may require thickening. This can be further assessed during construction.
- Fabric filter-encased curb subdrains are required on both sides of the road, connecting into a positive outlet, such as catch basins or manholes.

6.7 Soil Parameters

The recommended soil parameters for the project design are given in Table 5.

Table 5 – Soil Parameters

Unit Weight and Bulk Factor				
	Unit Weight (kN/m^3)		**Estimated Bulk Factor**	
	Bulk	**Submerged**	**Loose**	**Compacted**
Earth Fill / Silt	20.5	10.5	1.20	1.00
Silty Clay	21.0	11.0	1.30	1.00
Weathered Shale	23.5	13.5	1.35	1.10

Lateral Earth Pressure Coefficients			
	Active	**At Rest**	**Passive**
	K_a	**K_o**	**K_p**
Compacted Earth Fill / Silt	0.40	0.55	2.50
Silty Clay	0.50	0.65	2.00
Shale	0.25	0.40	4.00

Coefficients of Friction	
Between Concrete and Granular Base	0.5
Between Concrete and Sound Natural Soils	0.4

6.8 Excavation

Excavation should be carried out in accordance with Ontario Regulation 213/91. For excavation purposes, the types of soils are classified in Table 6.

Table 6 - Classification of Soils for Excavation

Material	Type
Weathered Shale and Silty Clay	2
Existing Earth Fill and dewatered Silt	3
Saturated Soils	4

In excavations below the saturated levels, the groundwater yield may be appreciable and likely persistent. The quantity will slow down after sometime. The groundwater can be collected in sumps and removed by conventional pumping.

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 sewer subgrade. 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 Wycliffe Homes, for review by their designated consultants, financial institutions, and government agencies. Use of this report is subject to the conditions and limitations of the contractual agreement. The material in the report reflects the judgement of Kin Fung Li, B.Eng., and Bennett Sun, 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, 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.

Kin Fung Li, B.Eng.

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YA/KFL/BS

LICENSED PROFESSIONAL ENGINEER
CHUN C. B. SUN
May 24, 2017
PROVINCE OF ONTARIO

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.: 1704-S067

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

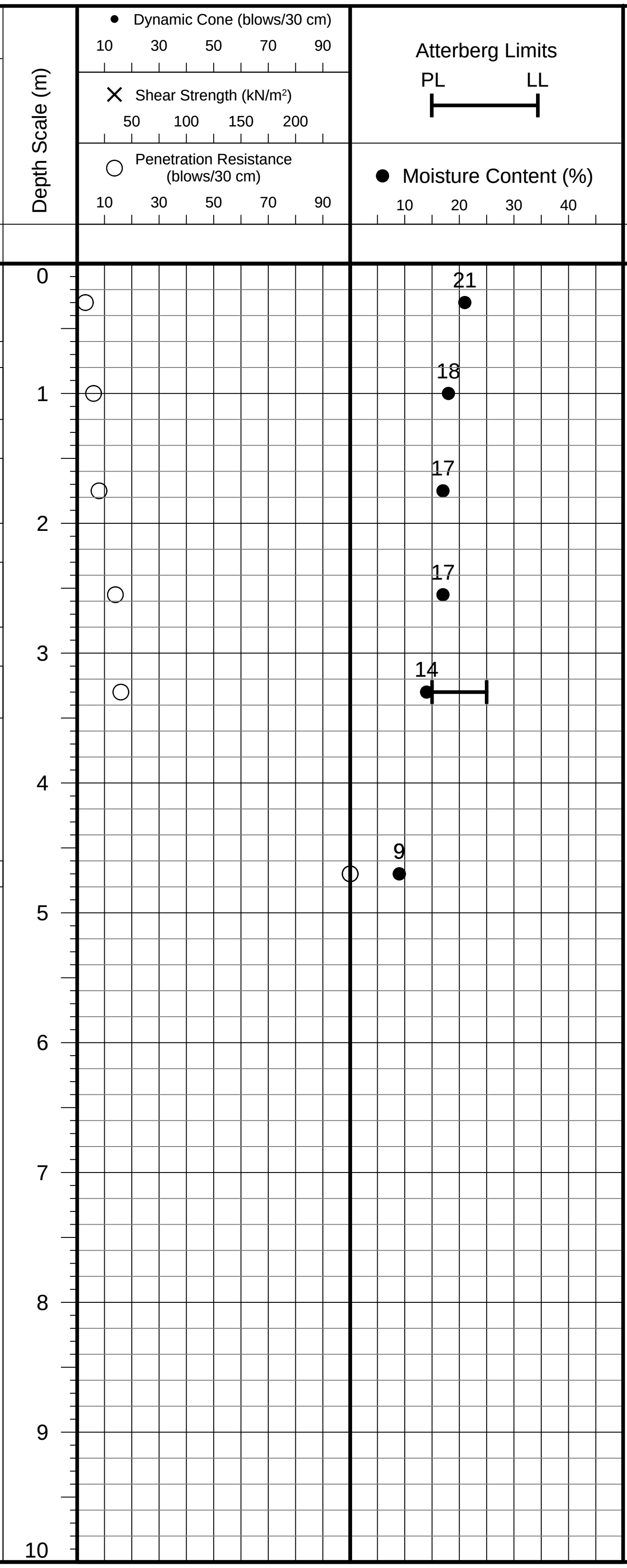
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight Auger

PROJECT LOCATION: 3171 Lakeshore Road West, Town of Oakville

DRILLING DATE: April 25, 2017

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
84.8	Ground Surface			
0.0	**10 cm TOPSOIL**			
	Brown **EARTH FILL** sand silt occ. rootlet and topsoil inclusions	1	DO	3
83.8 1.0	Brown, loose, wet **SILT** some clay a trace of sand	2	DO	6
		3	DO	8
82.5 2.3	Greyish brown, stiff to very stiff **SILTY CLAY**	4	DO	14
		5	DO	16
80.2 4.6	Reddish brown, weathered, stratified **SHALE**	6	DO	50/8
80.0 4.8	**END OF BOREHOLE** Auger and sample refusal			



WATER LEVEL

W.L. @ El. 82.8 m upon completion

Cave-In @ El. 80.2 m below grade upon completion

JOB NO.: 1704-S067

LOG OF BOREHOLE NO.: 2

FIGURE NO.: 2

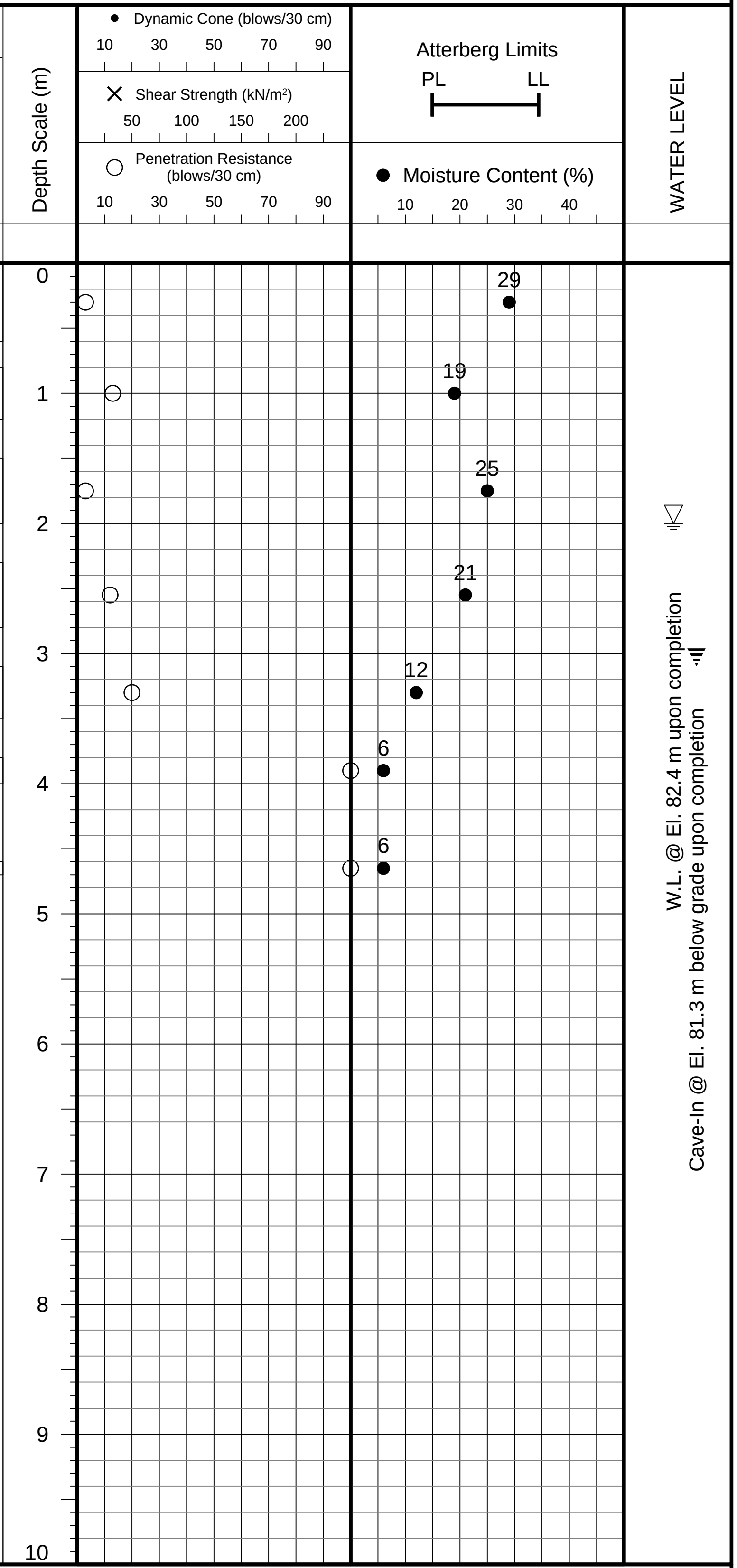
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight Auger

PROJECT LOCATION: 3171 Lakeshore Road West, Town of Oakville

DRILLING DATE: April 25, 2017

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
84.4	Ground Surface			
0.0	**30 cm TOPSOIL**			
	Brown **EARTH FILL** silty sand occ. rootlet and topsoil inclusions	1	DO	3
		2	DO	13
		3	DO	3
82.1 2.3	Brown, compact, wet **SILT** some clay a trace of sand	4	DO	12
81.1 3.3	Red, very stiff **SILTY CLAY** (clay-shale reversion)	5	DO	20
80.6 3.8	Reddish brown, weathered, stratified **SHALE**	6	DO	50/5
		7	DO	50/8
79.7 4.7	**END OF BOREHOLE** Auger and sample refusal			



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JOB NO.: 1704-S067

LOG OF BOREHOLE NO.: 3

FIGURE NO.: 3

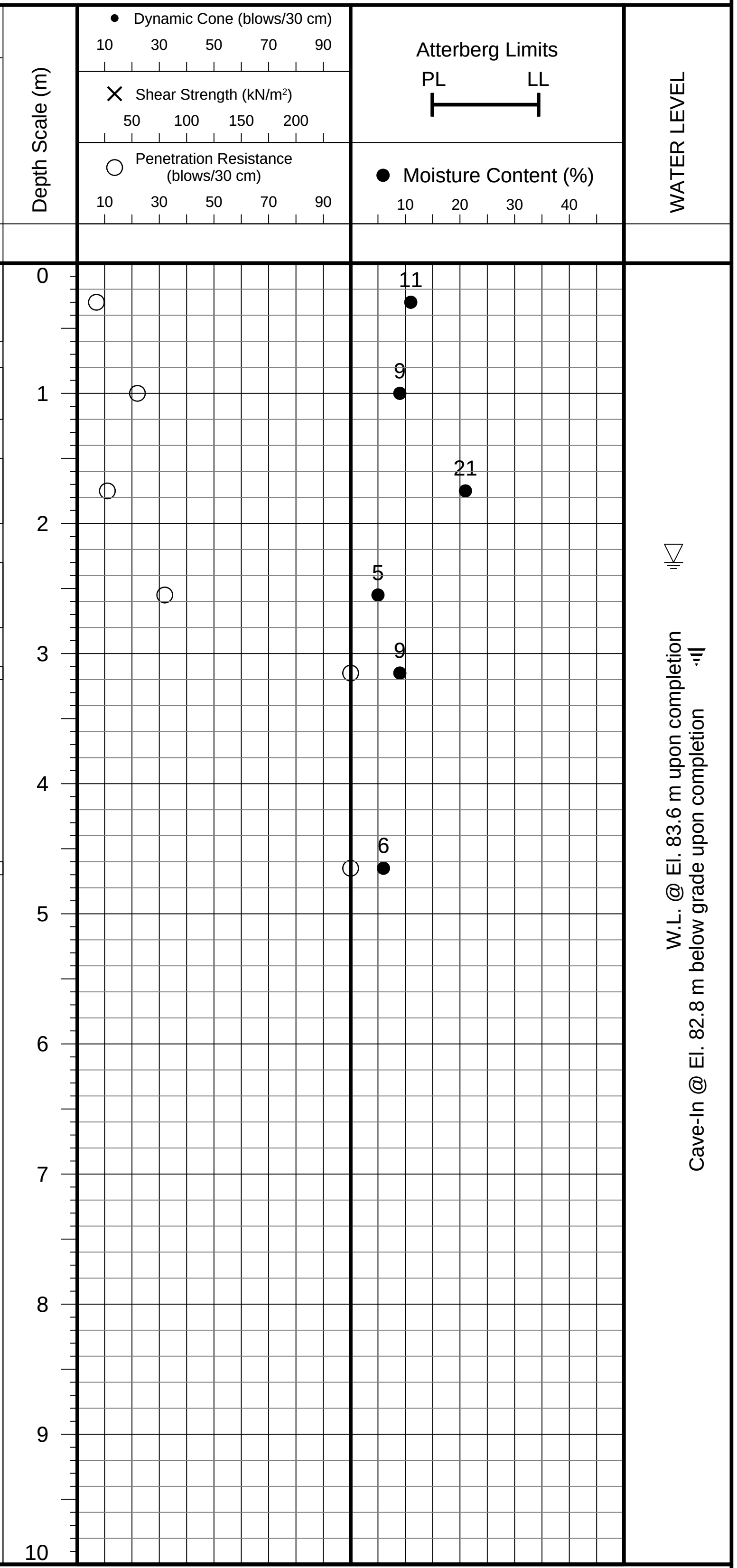
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight Auger

PROJECT LOCATION: 3171 Lakeshore Road West, Town of Oakville

DRILLING DATE: April 25, 2017

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES		
		Number	Type	N-Value
85.9	Ground Surface			
0.0	Brown **EARTH FILL** sand to silty sand	1	DO	7
		2	DO	22
84.4 1.5	Brown, stiff to hard **SILTY CLAY**	3	DO	11
		4	DO	32
83.2 2.7	Reddish brown, weathered, stratified **SHALE**	5	DO	50/10
		6	DO	50/10
81.2 4.7	**END OF BOREHOLE** Auger and sample refusal			



W.L. @ El. 83.6 m upon completion

Cave-In @ El. 82.8 m below grade upon completion

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JOB NO.: 1704-S067

LOG OF BOREHOLE NO.: 4

FIGURE NO.: 4

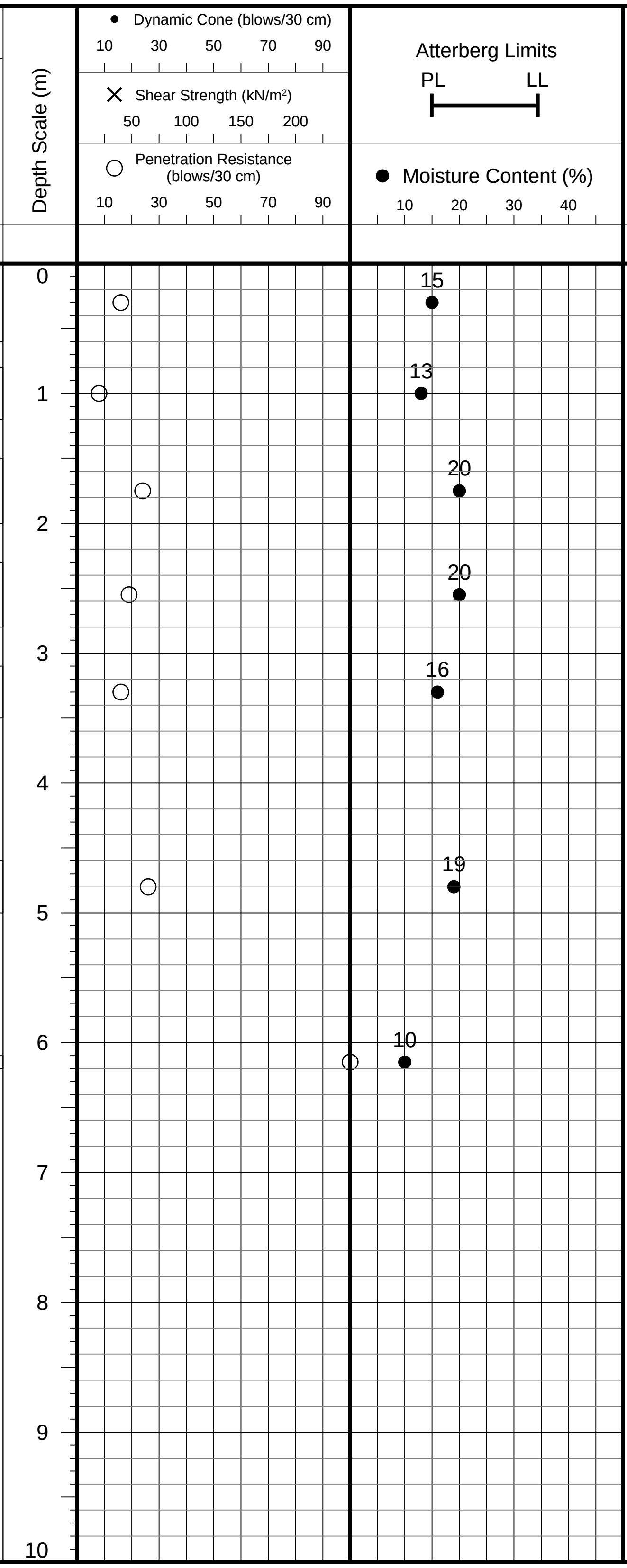
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight Auger

PROJECT LOCATION: 3171 Lakeshore Road West, Town of Oakville

DRILLING DATE: April 25, 2017

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES		
		Number	Type	N-Value
86.4	Ground Surface			
0.0	Brown **EARTH FILL** sand to silty sand rock fragment	1	DO	16
		2	DO	8
84.9 1.5	Brown, compact **SILT** some clay a trace of sand	3	DO	24
		4	DO	19
83.3 3.1	Greyish brown, very stiff **SILTY CLAY** wet silty sand layer	5	DO	16
		6	DO	26
81.5 4.9	Reddish brown, weathered, stratified **SHALE**	7	DO	50/3
80.2 6.2	**END OF BOREHOLE** Auger and sample refusal			



WATER LEVEL

Cave-In @ El. 84.7 m below grade upon completion

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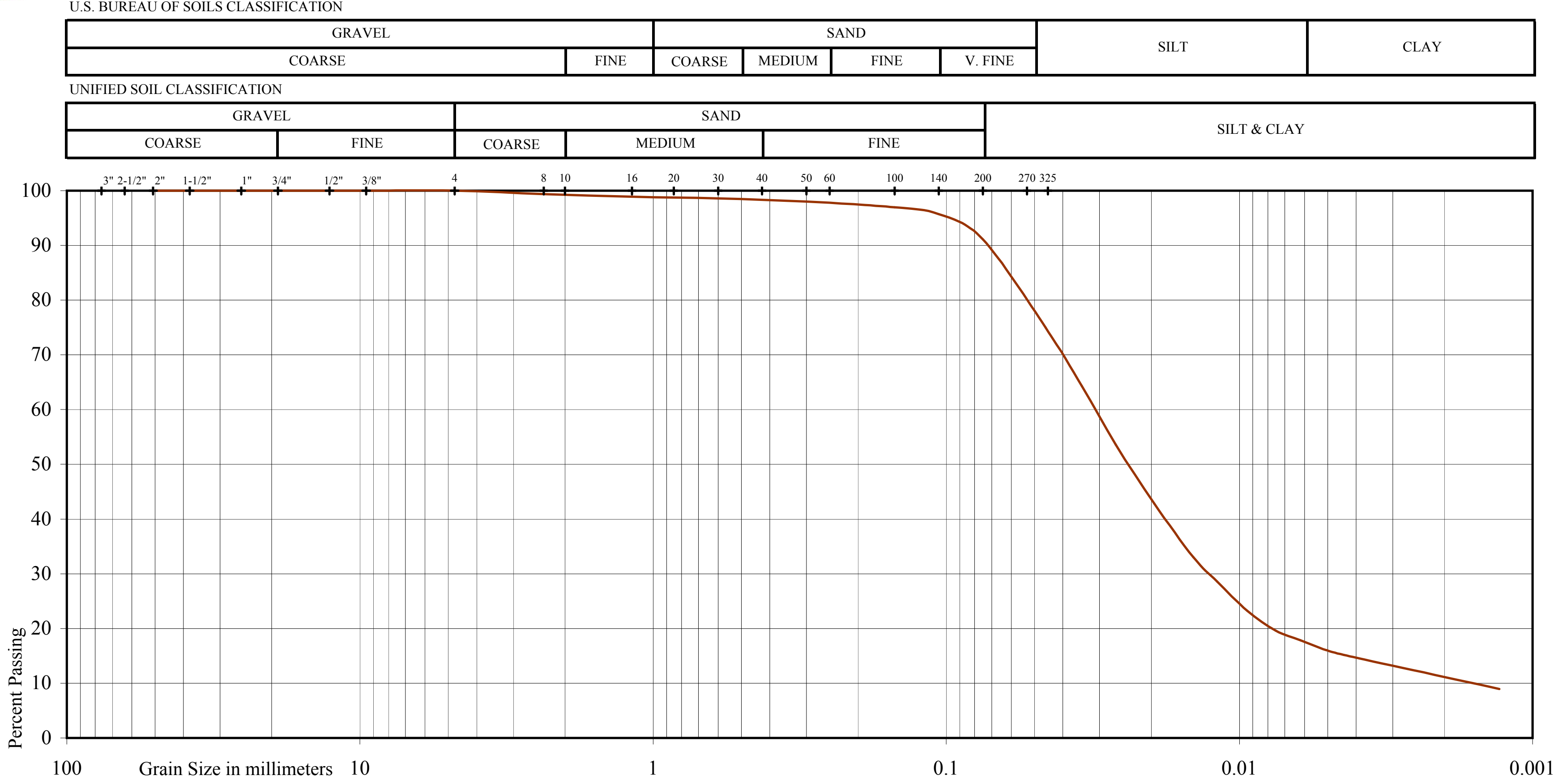
GRAIN SIZE DISTRIBUTION

Reference No: 1704-S067



Project: Proposed Residential Development

Location: 3171 Lakeshore Road West, Town of Oakville

Borehole No: 1

Sample No: 3

Depth (m): 1.8

Elevation (m): 83.0

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 17

Estimated Permeability

(cm./sec.) = 10^{-6}

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

Figure: 5

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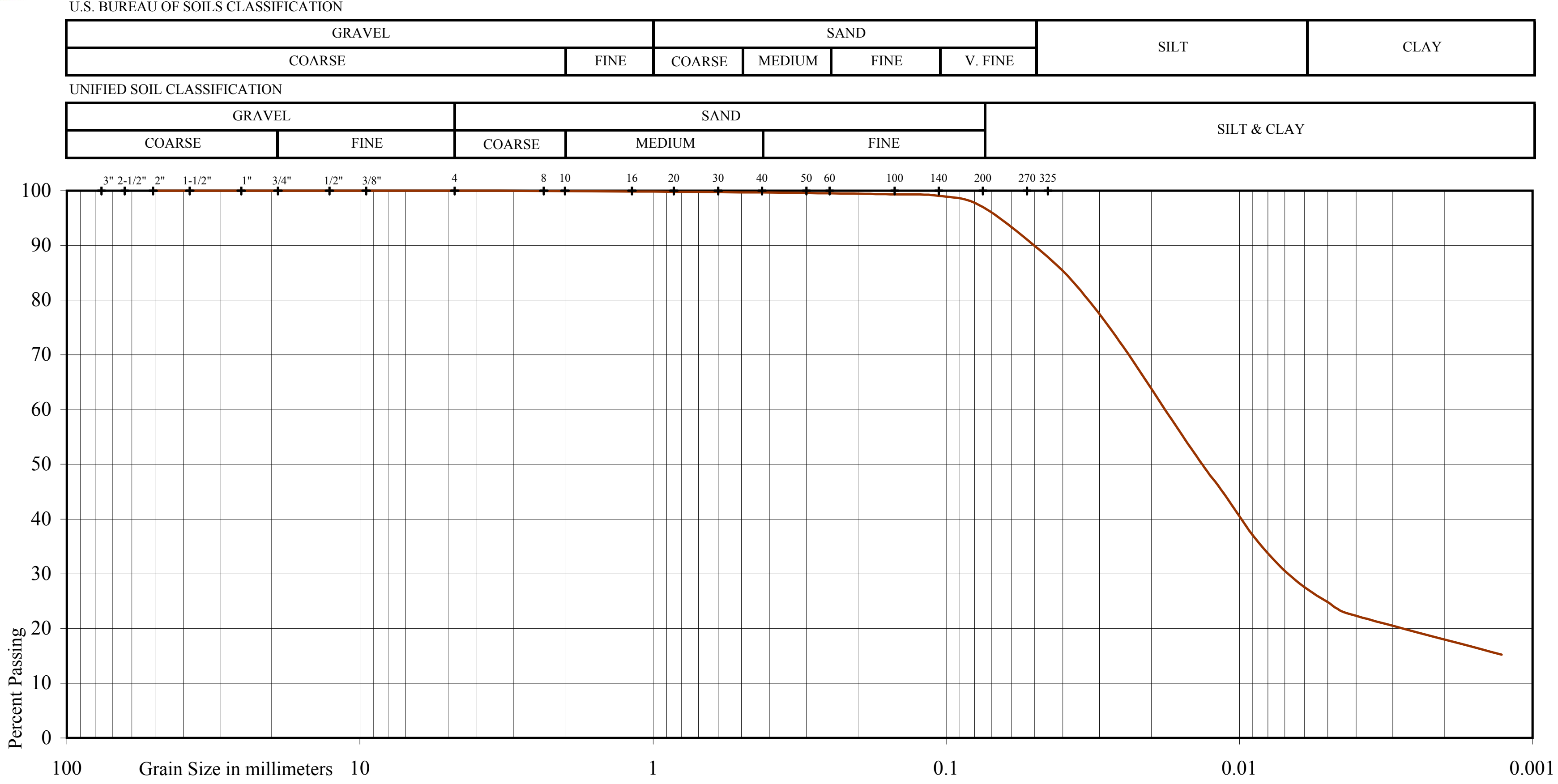
GRAIN SIZE DISTRIBUTION

Reference No: 1704-S067



Project: Proposed Residential Development

Location: 3171 Lakeshore Road West, Town of Oakville

Borehole No: 1

Sample No: 5

Depth (m): 3.3

Elevation (m): 81.5

Liquid Limit (%) = 25

Plastic Limit (%) = 15

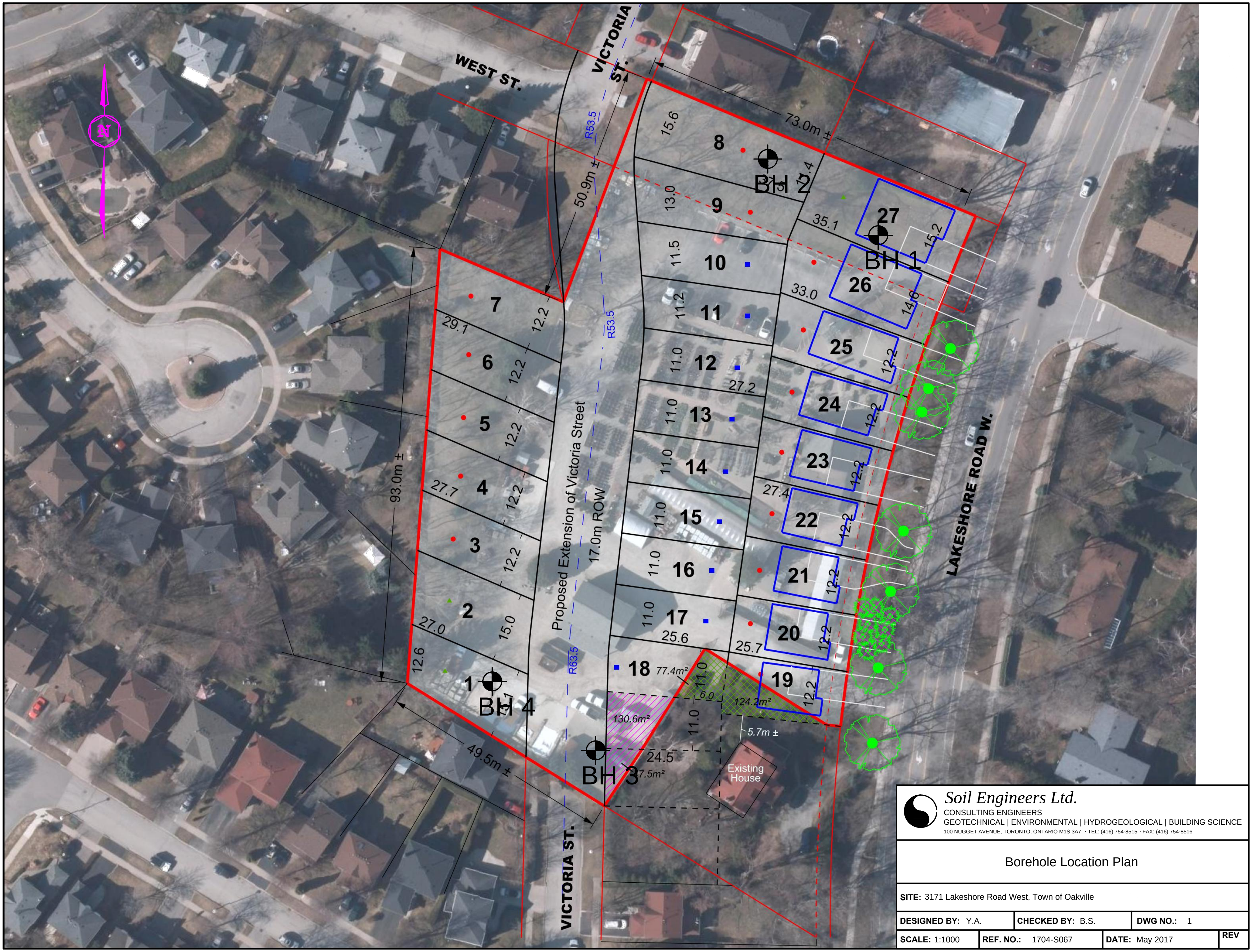
Plasticity Index (%) = 10

Moisture Content (%) = 14

Estimated Permeability (cm./sec.) = 10^{-7}

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

Figure: 6



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Borehole Location Plan

SITE: 3171 Lakeshore Road West, Town of Oakville

DESIGNED BY: Y.A.	**CHECKED BY:** B.S.	**DWG NO.:** 1

SCALE: 1:1000	**REF. NO.:** 1704-S067	**DATE:** May 2017	**REV**

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SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

JOB NO.: 1704-S067
REPORT DATE: May 2017
PROJECT DESCRIPTION: Proposed Residential Development

PROJECT LOCATION: 3171 Lakeshore Road West, Town of Oakville

LEGEND

▨ TOPSOIL
🟩 SILT
▨ SILTY CLAY
🟥 SHALE
▩ FILL

▽ WATER LEVEL (END OF DRILLING)
≡ CAVE-IN

BH No.:	1	2	3	4
El. (m):	84.8	84.4	85.9	86.4