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A REPORT TO
833697 ONTARIO LIMITED

A SOIL INVESTIGATION FOR PROPOSED
RESIDENTIAL DEVELOPMENT

1657 MAPLEVIEW ROAD

TOWN OF INNISFIL

Reference No. 0611-S047

DECEMBER 2006

DISTRIBUTION

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1.0 **INTRODUCTION**

In accordance with written authorization dated November 8, 2006, from Mr. Joran Weiner, Project Manager, of Armland Group, on behalf of 833697 Ontario Limited, a soil investigation was carried out at a 18.95 hectare parcel of land located at the municipal address of 1657 Mapleview Road, Town of Innisfil, for a proposed Residential Development.

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

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

2.0 SITE AND PROJECT DESCRIPTION

The site is situated in the Lake Simcoe basin, within the region known as Simcoe Lowlands, where the glacial till has been partly eroded in places by glacial Lake Algonquin and filled with gravel, sands (of glaciofluvial origin) and/or lacustrine sand, silt and clay. The sand and silt strata are likely to be part of the regional aquifer. In places, the aquifer may be under artesian and subterranean artesian pressure.

The investigated area consists mainly of a farm field with associated homestead and barns, with scattered clumps of trees and shrubs and a watercourse or a drainage ditch traversing the north portion of the property. The ground surface is relatively flat with undulations and generally descends towards Lake Simcoe to the east.

The property is bounded by Mapleview Road to the north. The neighbouring properties consist of agricultural lands.

The property will be subdivided into lots for residential usage. The new development will be provided with municipal services and roadways meeting the urban standards. At the time of preparation of this report, detailed site and grading plans were not available.

3.0 FIELD WORK

The field work, consisting of 10 boreholes to depths ranging from 4.7 to 6.6 m, was performed on November 20, 2006, at the locations shown on the Borehole Location Plan and Subsurface Profile, Drawing No. 1.

The holes 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 recorded by a Senior Geotechnical Technician.

A topographic plan of the subdivision was not available; therefore, the sampling depths and the depth 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 10, inclusive. The revealed stratigraphy is plotted on the subsurface profile, Drawing No. 1, and the engineering properties of the disclosed soils are discussed herein.

Beneath a veneer of topsoil, the site is generally underlain by strata of silty clay till, sandy silt till and silty sand till; lenses, layers and strata of silty clay, fine to medium sand and silty fine sand were occasionally encountered at various depths and locations.

4.1 Topsoil (All Boreholes)

The revealed topsoil ranges in thickness from 25 to 60 cm. It should be noted that the property consists of farmland, and the site is slightly undulating. The past cultivation will invariably have filled the localized depressions, and topsoil eroded by runoff will have been deposited in the low-lying areas. Therefore, thicker topsoil than that found in the boreholes may occur in places. This renders it difficult to estimate the quantity of topsoil to be stripped and a test pit program should be carried out to obtain more data for the estimate prior to the project construction. In order to prevent overstripping, diligent control of the stripping operation will be required.

The topsoil is dark brown in colour from which it is inferred that it contains an appreciable amount of roots and humus. These materials are unstable and compressible under loads; therefore, the topsoil is considered to be void of engineering value, but can be used for general landscaping purposes.

A fertility analysis should be carried out to assess the suitability of the topsoil for use as a planting soil or sodding medium.

Due to its high humus content, the topsoil may produce volatile gases and will generate an offensive odour under anaerobic conditions; 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 **Silty Clay Till** (All Boreholes, except Boreholes 2, 8 and 9)

The till was found dominating the soil stratigraphy and consists of a random mixture of soils; the particle sizes range from clay to gravel, with the clay fraction exerting the dominant influence on the soil properties. Occasional silt and sand seams and layers, and cobbles and boulders were also detected in the clay till mantle.

Hard resistance to augering was encountered, showing that occasional cobbles and boulders are embedded in the till mantle, particularly in the lower zone of the stratum.

Sample examinations show that the surficial zone, 1.0± m, and in places, 1.2± m below the prevailing ground surface, is permeated with fissures, indicating that it has been fractured by the weathering process.

The obtained 'N' values range from 4 to 100+, with a median of 26 blows per 30 cm of penetration, showing that the consistency of the till is firm to hard, being generally very stiff. The firm till was generally encountered in the weathered zone.

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

Liquid Limit	23%
Plastic Limit	15%
Natural Water Content	8% to 29% (median 14%)
	(generally decreasing with depth)

The above results show that the till is a cohesive material with low plasticity. The natural water content generally lies close to its plastic limit, confirming the consistency of the till determined by the 'N' values. The low 'N' values and high water content of 20%+ were generally obtained in the badly weathered zone, showing the clay till in this zone has been fractured by the weathering process, allowing infiltrating precipitation to wet the fissures, thus softening its consistency.

Grain size analyses were performed on 2 representative samples of the silty clay till; the results are plotted on Figure 11.

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

- Moderate frost susceptibility and water erodibility.

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

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

- A cohesive soil, its shear strength is primarily derived from consistency which is inversely related to its moisture content. It contains sand; therefore, its shear strength is augmented by internal friction.
- In stiff to hard till, the clay till is generally stable in a relatively steep cut; however, steep cuts in the firm till will slough readily.
- 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 3,500 ohm/cm.

4.3 **Sandy Silt Till** (Boreholes 2, 3 and 4)

The sandy silt till was encountered in 3 of the 10 boreholes and was found generally in the east central sector of the site, in the lower zone of the stratigraphy beneath the silty clay till and silty clay strata. It consists of a random mixture of soil particle sizes ranging from clay to gravel, with the silt being the predominant fraction. The material is unsorted, being heterogeneous, showing that it is a glacial till.

A tactile examination of the soil samples showed that the till contains occasional wet sand and silt layers. It is cemented and varies in cohesiveness from appreciable to slight, revealing that the till contains traces to some clay. The clay content varies with depth and location.

The natural water content values of the samples range from 8% to 13%, with a median of 9%, showing that the till is in a moist to very moist condition.

The relative density of the till, as inferred from the 'N' values of 35 to 50, with a median of 41, is dense to very dense, being generally dense.

The resistance to augering encountered during the site investigation shows that cobbles and boulders are present in the stratum, particularly in the lower zone of the stratum.

A grain size analysis was performed on 1 representative sample of sandy silt till; the gradation is plotted on Figure 12.

According to the field and laboratory findings, the deduced soil engineering properties pertaining to the project are listed below:

- High frost susceptibility and medium to low water erodibility.
- Moderately low permeability, with an estimated coefficient of permeability of 10^{-6} cm/sec, and runoff coefficients of:

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

- Its shear strength is density dependent in its in situ state, and is also augmented by cohesion and cementation. Due to its dilatancy, the saturated till mantle will suffer a reduction of shear strength when subjected to dynamic loads.
- The till will slough slowly if submerged in an unconfined state or from an open-face cut under seepage conditions, particularly in the zone where the saturated sand layers are prevalent. The sides will be stable with a relatively steep slope when excavated in a moist condition.
- A poor flexible pavement-supportive material, with an estimated CBR value of 7%.
- Moderate corrosivity to buried metal, with an estimated electrical resistivity of 4,500 ohm/cm.

4.4 **Silty Sand Till** (Boreholes 6 and 7)

The sand till was encountered beneath the silty clay till in 2 boreholes in the central to southwest sector of the property, extending at least to a depth of 6.3 m in Borehole 6 and overlying the silty fine sand in Borehole 7. It consists of a random mixture of particle sizes ranging from clay to gravel, with sand being the predominant fraction. Its structure is heterogeneous.

A tactile examination of the soil samples indicated that the till is slightly cemented, and it displayed some cohesion when wetted.

The relative density of the till, as inferred from the ‘N’ values of 20 to 100+, with a median of 38, is compact to very dense, being generally dense.

The resistance to augering encountered during the site investigation shows that occasional to numerous cobbles and boulders are present in the stratum and their amount increases with depth.

The natural water content values of the samples range from 8% to 13%, with a median of 10%, showing that the till is in a moist to very moist condition.

A grain size analysis was performed on 1 representative sample of the silty sand till; the gradation is plotted on Figure 13.

According to the above findings, the engineering properties are listed below:

- High frost susceptibility and moderate water erodibility.
- Moderate permeability, with an estimated coefficient of permeability of 10^{-6} cm/sec, and runoff coefficients of:

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

- A frictional soil, its shear strength is primarily derived from internal friction and is augmented by cementation. Therefore, its strength is soil density-dependent.
- It will be stable in steep cuts; however, under prolonged exposure, localized sheet collapse will likely occur.
- A fair pavement-supportive material, with an estimated CBR value of 10%.

- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 5,500 ohm/cm.

4.5 **Silty Clay** (Boreholes 1, 2 and 5)

The silty clay was encountered in 3 of the 10 boreholes in the north sector of the property and it occurs generally beneath the topsoil veneer, silty fine sand or fine to medium sand strata. It contains wet silt and fine sand layers and has a faintly varved structure, indicating that the clay is a lacustrine deposit.

The wet silt layers in the silty clay became highly dilatant when shaken. The overall strength of the clay was weakened when kneaded, showing its strength is susceptible to remoulding.

Sample examinations show that the surficial zone, 1.2± m below the prevailing ground surface, is permeated with fissures, indicating that it has been fractured by the weathering process.

The obtained 'N' values range from 7 to 36, with a median of 22, from which the consistency of the clay is inferred as firm to hard, being generally very stiff. The firm to stiff clay was generally encountered in the weathered zone.

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

Liquid Limit	37%
Plastic Limit	20%
Natural Water Content	17% to 32% (median 26%)

The above values show that the clay is a material of medium plasticity. The water content of the clay generally lies above its plastic limit, indicating that the clay and laminated silt and sand layers were wet.

In some instances the high water content values do not reflect the consistency of the clay as determined by the 'N' values. This is caused by the presence of the wet sand and silt layers in the clay mantle.

A grain size analysis was performed on 1 representative sample of the silty clay; the resulting gradation is plotted on Figure 14.

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

- High frost susceptibility, and due to the high silt content and the presence of the wet silt layers, high soil-adfreezing potential.
- Low to moderate water erodibility.
- The clay is virtually impervious. However, due to the sand and silt layers, the lateral permeability is higher than the vertical permeability. The estimated coefficient of permeability is 10^{-8} cm/sec, with 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 is inversely dependent on soil moisture. It will be susceptible to a reduction in strength if remoulded. The silt and sand (layers) are frictional soils. Their strength is soil density-dependent. The wet silt, due to its dilatancy, is susceptible to impact disturbance; i.e., the disturbance will induce a pore pressure build-up within the mantle, resulting in soil dilation and a reduction in shear strength.
- In excavations, the stiff to very stiff clay crust will be stable in a relatively steep cut for a short duration; however, as water seepage saturates the sand layers, the sides will slough, and sheet collapse may occur without warning.
- A very poor flexible pavement-supportive material, with an estimated CBR value of 3% or less.
- Moderately high corrosivity to buried metal, with an estimated electrical resistivity of 3,500 ohm/cm.

4.6 **Fine to Medium Sand** (Borehole 1)

The fine to medium sand was found in the upper zone of the stratigraphy beneath the topsoil and overlying the silty clay in Borehole 1. Its occurrence is considered to be localized in nature.

The sand contains a trace of gravel and some silt with subangular particle shapes, indicating that it is an interglacial fluvial deposit. The sand is non-cohesive and is found to be wet to saturated.

The obtained ‘N’ value of the fine to medium sand is 9, indicating that its relative density is loose. The loose condition occurs in the surficial layers which have been loosened by the weathering process.

The natural water content values were found to be 16% and 26%, confirming the wet to saturated condition disclosed by sample examination. Due to the pervious nature of the sand, some water may have drained during sampling; therefore, the lower water content value does not represent the true value.

A grain size analysis was performed on 1 representative sample of the fine to medium sand; the resulting gradation is plotted on Figure 15.

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

- Low frost susceptibility and high water erodibility.
- Pervious, with an estimated coefficient of permeability of 10^{-3} cm/sec, and runoff coefficients of:

Slope	
0% - 2%	0.04
2% - 6%	0.09
6% +	0.13

- A frictional soil, its shear strength is derived from internal friction and is, therefore, directly dependent on soil density.
- In cuts, the moist sand will be stable in a relatively steep slope, and the wet sand will slough readily. It will run with water seepage and boil under a piezometric head of 0.3 m.

- A fair material to support flexible pavement, with an estimated CBR value of 20%.
- Low corrosivity to buried metal, with an estimated electrical resistivity of 6,500 ohm/cm.

4.7 **Silty Fine Sand** (Boreholes 5, 7, 8, 9 and 10)

The silty fine sand deposit was encountered below the topsoil veneer or beneath the silty clay till and silty sand till in the lower zone of the stratigraphy in 5 of the 10 boreholes.

The silty fine sand contains occasional sandy silt and fine sand layers. The sorted structure indicates that it is a glaciolacustrine deposit.

The obtained 'N' values of the silty fine sand range from 5 to 100+, with a median of 32, indicating that the relative density of the deposit is loose to very dense, generally being dense. The loose silty fine sand occurred within a shallow depth, within 1.2± m from the prevailing ground surface which has been loosened by weathering.

The obtained natural water content values range from 10% to 23%, with a median of 19%, showing that the sand is in a wet to saturated condition. The wet silty fine sand layers became dilatant when shaken.

A grain size analysis was performed on 1 representative sample of the silty fine sand; the gradation is plotted on Figure 16.

According to the above findings, the soil engineering properties relating to the project are given below:

- A soil of high capillarity, with high water-retention capability.
- High frost susceptibility, with high soil-adfreezing potential.
- High water erodibility; it will migrate through small openings under low to moderate seepage pressure.
- Relatively pervious, with an estimated coefficient of permeability of 10^{-4} cm/sec, and runoff coefficients of:

Slope	
0% - 2%	0.07
2% - 6%	0.12
6% +	0.18

- Its shear strength is derived from internal friction, thus being density dependent. Due to its dilatancy, dynamic loads imposed on the saturated soil will render a reduction of its shear strength.
- When excavated, the soil will run with seepage and the bottom will boil under a piezometric head of 0.4 m.
- A fair pavement-supportive material, with an estimated CBR value of 8%.
- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 5,000 ohm/cm.

4.8 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 Till	8 to 29 (median 14)	15	11 to 20
Sandy Silt Till	8 to 13 (median 9)	11	7 to 16
Silty Sand Till	8 to 13 (median 10)	11	7 to 16
Silty Clay	17 to 32 (median 26)	19	14 to 24
Fine to Medium Sand	16 and 26	11	5 to 16
Silty Fine Sand	10 to 23 (median 19)	11	6 to 16

According to the above findings, the majority of the in situ soils are generally suitable for a 95% or + Standard Proctor compaction, while some of the weathered soils, silty clay and sands are excessively wet and will require prior aeration. This should be carried out during the dry, warm weather by spreading them thinly on the ground. The wet sands can be drained of excess water by being properly stockpiled for proper compaction. The overall water content of the sound tills is too dry or is on the dry side of the optimum. Wetting of these soils will be required, particularly in dry, warm weather and in areas where compaction is best performed on the wet side of the optimum. The soils having a low water content can be mixed with wetter soils, or can be wetted prior to or during structural compaction.

The silty clay and tills should be compacted using a heavy-weight, kneading-type roller. The sands can be compacted by a smooth roller with or without vibration, depending on the water content of the soils 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 hard silty clay till and the cemented, dense to very dense sandy silt till and silty sand till 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.

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, on the other hand, 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.

One should be aware that, with considerable effort, a 90%± Standard Proctor compaction of the wet sand and silt 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 soil mantle has increased to over 95% of its maximum Standard Proctor dry density.

The presence of boulders will prevent transmission of the compactive energy into the underlying material to be compacted. If an appreciable amount of boulders over 15 cm in size is mixed with the material, it must either be sorted or must not be used for construction of engineered fill and/or structural backfill.

5.0 GROUNDWATER CONDITIONS

The boreholes were checked for the presence of groundwater or the occurrence of cave-in upon their completion and monitored again after completion of the field work. The data are plotted on the Borehole Logs and listed in Table 2.

Table 2 - Groundwater Levels

BH No.	Borehole Depth (m)	Soil Colour Changes Brown to Grey Depth (m)	Seepage Encountered During Augering Depth (m)	Seepage Encountered During Augering Amount	Measured Groundwater/ Cave-in* Level on Completion Depth (m)
1	5.0	2.3	0.9	Appreciable	0.9/1.1*
2	6.6	1.5	0.9	Moderate	4.6
3	6.6	4.0	0.6	Moderate	0.2/4.0*
4	5.0	2.5	0.9	Some	1.8
5	5.0	3.9	1.2	Moderate	0.2/2.1*
6	6.3	2.0	3.1	Some	1.8
7	6.6	4.6	6.1	Moderate	2.7/3.1*
8	4.9	4.9+	1.0	Appreciable	0.9*
9	4.7	4.7+	1.5	Appreciable	1.4*
10	6.6	6.6+	3.1	Appreciable	2.7/4.3*

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

The ground surface in many areas was wet at the time of the investigation, showing recent precipitation has saturated the ground.

Groundwater and groundwater seepage were detected in all of the boreholes, at depths ranging from 0.2 to 6.1 m from the prevailing ground surface, in the soil fissures and in the sand and silt layers in the tills and in the sand deposits. The groundwater drains easterly. Boreholes 3, 4, 6, 7 and 10 were ponded at the surface due to recent heavy precipitation which saturated the boreholes in the low-lying areas.

The soil colour changes from brown to grey at depths of 1.5± to 6.6± m from the prevailing ground surface. The brown soils in the upper zone have oxidized and the permanent groundwater occurs in the grey zone. The groundwater seepage encountered in the brown soil zone is infiltrated precipitation perched in the voids in the sand and silt seams and layers laminated in the clay and tills and in the shallow sand deposit.

The groundwater yield from the tills and clay, due to their low permeability, is expected to be small to some and can be controlled by pumping from sumps; from the water-bearing sands and sand till deposits, depending on their extent, the yield will be moderate to appreciable and persistent.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation disclosed that beneath a veneer of topsoil, the site is generally underlain by strata of firm to hard, generally very stiff silty clay till, dense to very dense, generally dense sandy silt till and compact to very dense, generally dense silty sand till; lenses, layers and strata of firm to hard, generally very stiff silty clay, loose fine to medium sand and loose to very dense, generally dense silty fine sand were occasionally encountered at various depths and locations.

The loose and firm deposits are generally restricted to the weathered zone which occurs within a depth of 1.2± m from the prevailing ground surface.

The groundwater regime lies at a depth of 0.2± to 4.3± m below the prevailing ground surface in the water-bearing sand and drains in an easterly direction towards the watercourse. The groundwater level will fluctuate with seasons.

Perched groundwater derived from infiltrated precipitation will occur at shallower depths in wet seasons. The water yield from the tills and silty clay will be small to some, whereas the yield from the sand and sand till deposits will be moderate to appreciable and persistent, depending on the size and continuity of the deposits.

The geotechnical findings which warrant special consideration are presented below:

1. The thickness of the revealed topsoil is 25 to 60 cm. However, topsoil thicker than that disclosed by the boreholes is likely to occur in the low-lying depressions. The topsoil contains an appreciable amount of humus and will

generate volatile gases under anaerobic conditions; therefore, it should not be buried within the building envelope or deeper than 1.2 m below the exterior finished grade of the subdivision. The topsoil should therefore be stripped for the project construction.

2. The surficial soils are generally weathered in the zone extending to a depth of 1.2± m from the prevailing ground surface. These soils are weak and will consolidate under surcharge loads; therefore, the footing subgrade must be inspected to ensure that its condition is compatible with the design of the foundations.
3. Extensive cut and fill will likely be required for the site grading and it is generally more economical to place engineered fill for normal footing, sewer and road construction.
4. In the areas where water-bearing sands occur, the sides of the excavations will run and the bottom will boil under a piezometric head of about 0.3 to 0.4 m in excavations in the wet sand strata. The excavation must be stabilized by vigorous pumping from closely spaced sump-wells or, if necessary, by a well-point dewatering system, depending on the depth of excavation. Therefore, the appropriate method of dewatering should be determined by a test pit programme carried out by the contractor prior to tendering and construction of the project.
5. As shown, the groundwater in many areas lies at a relatively shallow depth. Depending on the designed grade of the basement, floor subdrains may be required for protection against hydrostatic uplift and groundwater upfiltration. This can be further assessed at the time of basement excavation.
6. Due to the occurrence of shallow groundwater in the water-bearing sand, an intercepting subdrain system may be required. This can be further assessed

when the grading of the site has been prepared in order to minimize the impact of the groundwater to the proposed development.

7. As noted, the tills contain occasional boulders. Extra effort and a properly equipped appropriate backhoe will be required for excavation. Boulders larger than 15 cm in size are not suitable for structural backfill.
8. In the water-bearing sand, the bedding material for the underground services must consist of 20-mm Crusher-Run (graded) Limestone, or a Class ‘A’ bedding. In the tills plagued with water-bearing sand and silt layers, the bedding material should consist of 20-mm Crusher-Run (graded) Limestone.
9. Some portions of the disclosed soils are too wet for a 95% or + Standard Proctor compaction. In order to reduce the cost impact, the construction should preferably be scheduled so that the soils can be appropriately aerated.
10. Perimeter subdrains and dampproofing of the foundation walls will be required for basement construction. The subdrains should be shielded by a fabric filter to prevent blockage by silt.
11. In order to alleviate the problem of soil adfreezing, special measures should be implemented for the houses constructed in the areas where the groundwater regime is at a shallow depth and in the areas where the silty clay and silt occur.
12. Curb subdrains may be required by the municipality for road construction.
13. The existing water course or drainage ditch appears to be a relief of local surface runoff. It should be properly addressed by the Municipal Engineer planning the design of the residential development.

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 and Basement Construction

At the time of preparation of this report, site and grading plans for the project had not yet been determined. Based on the borehole findings, Maximum Allowable Soil Pressures of 150 kPa, 200 kPa and 400 kPa are recommended for the design of house and/or mid-rise building foundations founded on sound natural soil.

For normal house foundations, a Maximum Allowable Soil Pressure of 150 kPa can be used for the design of spread and strip footings founded on sound natural soil, 1.0± m to 2.0± m below the prevailing ground surface, or on engineered fill.

If mid-rise buildings are considered, Maximum Allowable Soil Pressures of 200 kPa and 400 kPa are recommended for the design of the normal spread and strip foundations. The suitable founding levels are listed in Table 3.

Table 3 - Founding Levels

	Maximum Allowable Soil Pressures and Suitable Founding Levels on Sound Natural Soil		
	150 kPa	**200 kPa**	**400 kPa**
Borehole No.	**Depth (m)**	**Depth (m)**	**Depth (m)**
1	2.0 or +	2.3 or +	-
2	1.2 or +	-	-

Table 3 - Founding Levels (Cont'd)

Borehole No.	Maximum Allowable Soil Pressures and Suitable Founding Levels on Sound Natural Soil		
	150 kPa	**200 kPa**	**400 kPa**
	Depth (m)	**Depth (m)**	**Depth (m)**
3	1.2 or +	-	5.5 or +
4	1.0 or +	1.5 or +	2.3 or +
5	1.2 or +	1.5 or +	-
6	1.0 or +	1.5 or +	5.0 or +
7	1.0 or +	1.5 or +	-
8	1.2 or +	1.5 or +	2.3 or +
9	1.2 or +	1.5 or +	2.3 or +
10	1.0 or +	1.5 or +	5.0 or +

If caissons are considered, a Maximum Allowable Soil Pressure of 600 or + kPa can be used for the design of caissons embedded at least 1.0 m into the soils with 'N' values of 30 to 100+. However, additional boreholes must be carried out to confirm and/or elaborate on the above recommendations when the building layout is determined.

The recommended soil pressures for the foundations incorporate a safety factor of 3 against shear failure of the underlying soil. The total and differential settlements of the footings on the soil are estimated to be 25 mm and 15 mm, respectively.

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

The footings should meet the requirements specified by the Ontario Building Code, and the structure should be designed to resist a minimum earthquake force calculated using the following:

F - Foundation Factor	1.0
v - Zonal Velocity Ratio	0.05

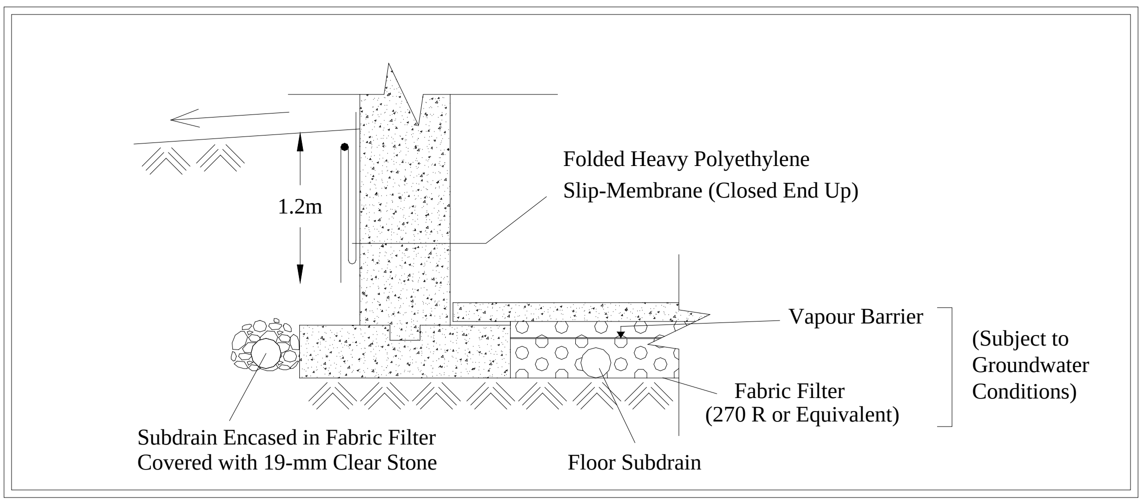
Due to the presence of topsoil and weathered soils, the footing subgrade must be inspected by a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, or a building inspector who has geotechnical experience, to ensure that the subgrade conditions are compatible with the foundation design requirements.

If engineered fill is required for the site development, a Maximum Allowable Soil Pressure of 150 kPa can be used for the design of foundations on engineered fill. The procedures and requirements for engineered fill are discussed in Section 6.2.

In order to minimize the impact of hydrostatic pressure and uplift, which incurs a high cost for the construction of a properly waterproofed basement, the basement level should be at a depth at least 0.6 m above the detected groundwater elevation. The walls must be dampproofed (assuming the basement level is at least 0.6 m above the groundwater regime) or waterproofed, if required. Subdrains consisting of a filter-wrapped weeper must be installed beneath the basement floor and connected to a sump-well. A vapour barrier must be placed in the granular base of the floor above the crown of the subdrain. The above recommendations should be further assessed and/or confirmed during construction.

As previously discussed, the silty clay and silty fine sand 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 recommended measures are schematically illustrated in Diagram 1.

Diagram 1 - Frost Protection Measures (Foundation)



6.2 Engineered Fill

Where earth fill is required to raise the site, the engineering requirements for a certifiable fill for road construction, municipal services, slab-on-grade and footings designed with a 150 kPa Maximum Allowable Soil Pressure are presented below:

1. All of the topsoil must be removed. The badly weathered soils must be subexcavated, sorted free of topsoil inclusions and deleterious material, aerated and properly compacted to at least 98% of their maximum Standard

Proctor dry density. The subgrade surface must be inspected and proof-rolled prior to any fill placement.

2. 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 lot grade and/or road subgrade. The soil moisture must be properly controlled on the wet side of the optimum. 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.
3. If imported fill is to be used, the hauler is responsible for its environmental quality and must provide a document to certify that it is free of hazardous contaminants.
4. 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.
5. The engineered fill must extend over the entire graded area, and the fill envelope must be clearly and accurately defined in the field and 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.
6. 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.

7. 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.
8. Where the fill is to be placed on sloping ground steeper than 1 vertical: 3 horizontal, the face of the sloping ground must be flattened to 3 + so that it is suitable for safe operation of the compactor and the required compaction can be obtained.
9. The fill operation must be inspected on a full-time basis by a technician under the direction of a geotechnical engineer.
10. The footing 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.
11. Any excavation carried out in the certified engineered fill must be reported to the geotechnical consultant who inspected the fill placement in order to document the locations of the 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.
12. Despite stringent control in the placement of the engineered fill, variations in the 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 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 Garages, Driveways and Landscaping

Due to the moderate to 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 garages should be backfilled with non-frost-susceptible granular material, with a frost taper at a slope of 1 vertical:1 horizontal.

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 1.5 m thick, with proper drainage, which will prevent water from ponding in the granular base. The in situ sand with a silt content of 10% or less is suitable for use as granular base.

6.4 Underground Services

The subgrade for the underground services should consist of properly compacted inorganic earth fill or sound natural soils. A Class ‘B’ bedding is recommended for the design of the underground services construction. The bedding material should consist of compacted 20-mm Crusher-Run Limestone, or equivalent. Lean concrete and thickening of the Crusher-Run Limestone bedding may be used for subgrade stabilization. In areas where extensive dewatering is required, a Class ‘A’ concrete bedding may be necessary.

In wet sands the pipe joints should be leak-proof or they should be provided with a waterproof membrane. This is to prevent migration of fines due to leaks from faulty joints.

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 by a fabric filter to prevent silting.

6.5 Trench Backfilling

The backfill in the 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; within 0.6 m from the subgrade, the compaction should be increased to 98% of the respective maximum Standard Proctor dry density to provide the required stiffness for pavement construction.

The in situ inorganic soils are suitable for use as trench fill; however, the water content of the some of the occurring soils, as determined, is generally too wet for a 95% or + Standard Proctor compaction. The soils can be aerated or the wet sand can be drained by proper stockpiling prior to structural compaction. Furthermore, some of the silty clay till is too dry and will require the addition of water prior to backfilling.

In normal construction practice, the problem areas of road 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.

The narrow trenches for services crossings should be cut at 1 vertical:2 horizontal so that the backfill in the trenches can be effectively compacted. Otherwise, soil arching in the trenches will prevent the achievement of proper compaction. In this case, imported sand fill which can be appropriately compacted by using a smaller vibratory compactor must be used. The areas at the interface of the native soil and the sand backfill should preferably be flooded for at least 1 day.

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 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, seepage collars should be provided.

6.6 Pavement Design

According to the borehole findings, the recommended pavement design for local roads is given in Table 4.

Table 4 - Pavement Design

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder Local Collector	 50 65	HL-8
Granular Base	150	20-mm Crusher-Run Limestone

Table 4 - Pavement Design (Cont'd)

Course	Thickness (mm)	OPS Specifications
Granular Sub-base Local Collector	 500 600	Granular 'B'

The granular base and sub-base should be compacted to 100% of the maximum Standard Proctor dry density.

In order to provide a stable subgrade for pavement construction, it is imperative that the subgrade within the 1.0 m zone below the underside of the granular base be compacted to at least 98% of its maximum Standard Proctor dry density with the moisture content 2% to 3% drier than the optimum. This is to provide adequate stability for the pavement construction.

The following measures should 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.
- Lot areas adjacent to the roads should be properly graded to prevent ponding of large amounts of water. Otherwise, the water will seep into the subgrade mantle and induce a regression of the subgrade strength with costly consequences for the pavement construction.
- Prior to placement of the granular bases, the subgrade should be proof-rolled and any soft spots should be rectified.

- If the roads are to be constructed during wet seasons, thickening of the granular sub-base may be required. The requirement for this can be determined at the time of road construction.
- Fabric filter-encased curb subdrains will be required. They should be installed in wet areas at depths below the underside of the granular sub-base.

6.7 Soil Corrosivity

The subgrade of the water main and the backfill material will consist generally of silty clay, silty clay till and silty fine sand material. They have moderately low to moderately high corrosivity to ductile iron pipes and metal fittings; therefore, cathodic protection will be required. The precise size of the anodes can be calculated at the time of the sewer construction when the soils at the water main level are exposed and sufficient samples can be taken to analyze their corrosivity potential. To estimate the anode weight requirements, the electrical resistivity which has been determined for each of the disclosed soils can be used.

6.8 Timing of Construction

As noted, the sand is highly water erodible. Therefore, bare surfaces should be sodded immediately, particularly where the surface grade is steeper than 5%.

The sod can effectively protect the soil against water erosion only if the roots are allowed to become properly secured in the underlying soils. This dictates that the sodding, at the latest, should be carried out by late September. Otherwise, the sod may be washed out during the wet seasons, resulting in serious soil erosion of sloping ground, and costly rectification would be unavoidable.

6.9 Fence Footings

The footings for the acoustic fence posts and piers must be embedded into the engineered fill or sound natural soil. A Maximum Allowable Soil Pressure of 150 kPa is recommended for the design of the footings and the soil parameters listed in Section 6.10 are recommended for the calculation of the lateral resistance.

6.10 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	**Loose**	**Compacted**
Weathered Tills	21.0	1.20	1.00
Sound Tills	22.0	1.33	1.05
Silty Clay	20.5	1.30	0.98
Sands	20.5	1.20	1.00
Lateral Earth Pressure Coefficients			
	Active K_a	**At Rest K_o**	**Passive K_p**
Sandy Silt Till and Silty Sand Till	0.33	0.43	3.00
Silty Clay Till and Silty Clay	0.40	0.50	2.50
Silty Fine Sand	0.33	0.43	3.00
Fine to Medium Sand	0.30	0.40	3.33

Table 5 - Soil Parameters (Cont'd)

Coefficients of Friction		
Between Concrete and Granular Base		0.60
Between Concrete and Sound Natural Soil		0.40
Maximum Allowable Soil Pressure For Thrust Block Design		
Engineered Fill and Sound inorganic Soils	100 kPa	

6.11 **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
Sound Tills	2
Weathered Tills, moist and properly dewatered Sand and Silt	3
Water-bearing Sand and Silt	4

In the tills and clay which are plagued with fissures and saturated sand and silt seams and layers, the sides of excavations above groundwater may suffer localized sloughing or side collapse. Therefore, the sides must be sloped at 1 vertical: at least 1 horizontal for stability.

At depths below the groundwater level, seepage in the clay and till mantles during excavation is expected to be slow and controllable by normal pumping from sumps.

Where excavations are to be carried out in the water-bearing sands, the possibility of flowing sides and bottom boiling dictates that the ground be predrained, either by pumping from closely spaced sump-wells or, if necessary, by the use of a well-point dewatering system. In order to provide a stable subgrade for the services or foundation construction, the groundwater should be depressed to at least 0.5 m below the subgrade. Alternatively, a sheeting structure can be installed around the excavation. The sheeting structure should be driven to a depth below the bottom of the excavation at least equal to the height of water above the bed of excavation. The sheeting structure must be properly designed to sustain the earth pressure, hydrostatic pressure and applicable surcharge.

As previously discussed, the groundwater yield from the sand deposits will be moderate to appreciable and persistent, depending on their extent and continuity; however, this should be confirmed by test pumping at the time of construction.

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

It should be noted that a Phase I Environmental Site Assessment has been carried out, and the assessment and recommendations will be presented under separate cover shortly. Therefore, this report deals only with a study of the geotechnical aspects of the proposed project.

This report was prepared by Soil Engineers Ltd. for the account of 833697 Ontario Limited, and for review by their designated consultants and government agencies. The material in it reflects the judgement of Eric Fron, Civ.Tech., and Victor S. Chan, 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.

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EF/VSC:jp

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

Soil Engineers Ltd.
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GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE

JOB NO.: 0611-S047

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

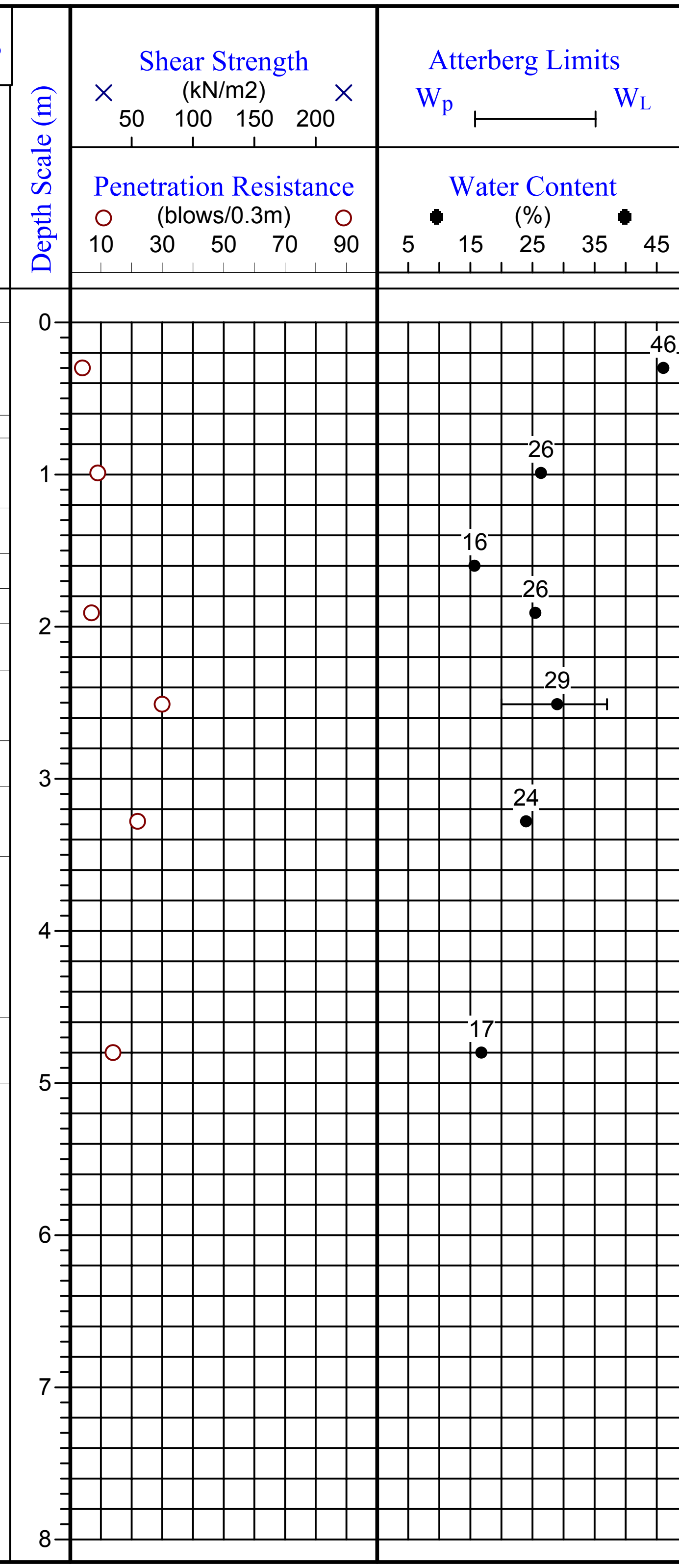
JOB DESCRIPTION: Proposed Residential Development

JOB LOCATION: 1657 Mapleview Road, Town of Innisfil

METHOD OF BORING: Flight-Auger

DATE: November 20, 2006

Elev. Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
	Ground Surface			
0.0	Dark brown **60cm TOPSOIL**	1	DO	4
	Brown, loose **FINE TO MEDIUM SAND** some silt occ. fine sand layers	2	DO	9
		3A	DO	-
1.8	Firm to very stiff brown / grey **SILTY CLAY** with wet silt and sand layers	3B	DO	7
		4	DO	30
		5	DO	22
4.2	Grey, stiff **SILTY CLAY, Till**	6	DO	14
5.0	**END OF BOREHOLE**			



WATER LEVEL

Cave-In @ depth of 1.1 m below prevailing ground surface on completion

W.L. @ depth of 0.9 m below prevailing ground surface on completion

Soil Engineers Ltd.

JOB NO.: 0611-S047

LOG OF BOREHOLE NO.: 2

FIGURE NO.: 2

JOB DESCRIPTION: Proposed Residential Development

JOB LOCATION: 1657 Mapleview Road, Town of Innisfil

METHOD OF BORING: Flight-Auger

DATE: November 20, 2006

Elev. Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Water Content (%)
	Ground Surface				
0.0	Dark brown, peaty **TOPSOIL**	1	DO	6	55
0.6	Stiff to very stiff **SILTY CLAY** fine sand layer	2A	AS	-	23
	weathered	2B	DO	8	26
	brown / grey	3	DO	14	17
		4	DO	22	23
		5	DO	17	31
	with wet silt and sand layers	6	DO	19	32
5.5	Grey, dense **SANDY SILT, Till** traces of gravel and clay occ. cobbles, boulders and wet sand and silt seams and layers	7	DO	35	8
6.6	**END OF BOREHOLE**				

Depth Scale (m): 0, 1, 2, 3, 4, 5, 6, 7, 8

Shear Strength (kN/m2): 50, 100, 150, 200

Penetration Resistance (blows/0.3m): 10, 30, 50, 70, 90

Atterberg Limits: W_p, W_L

Water Content (%): 5, 15, 25, 35, 45

WATER LEVEL

W.L. @ depth of 4.6 m below prevailing ground surface on completion

Soil Engineers Ltd.

JOB NO.: 0611-S047

LOG OF BOREHOLE NO.: 3

FIGURE NO.: 3

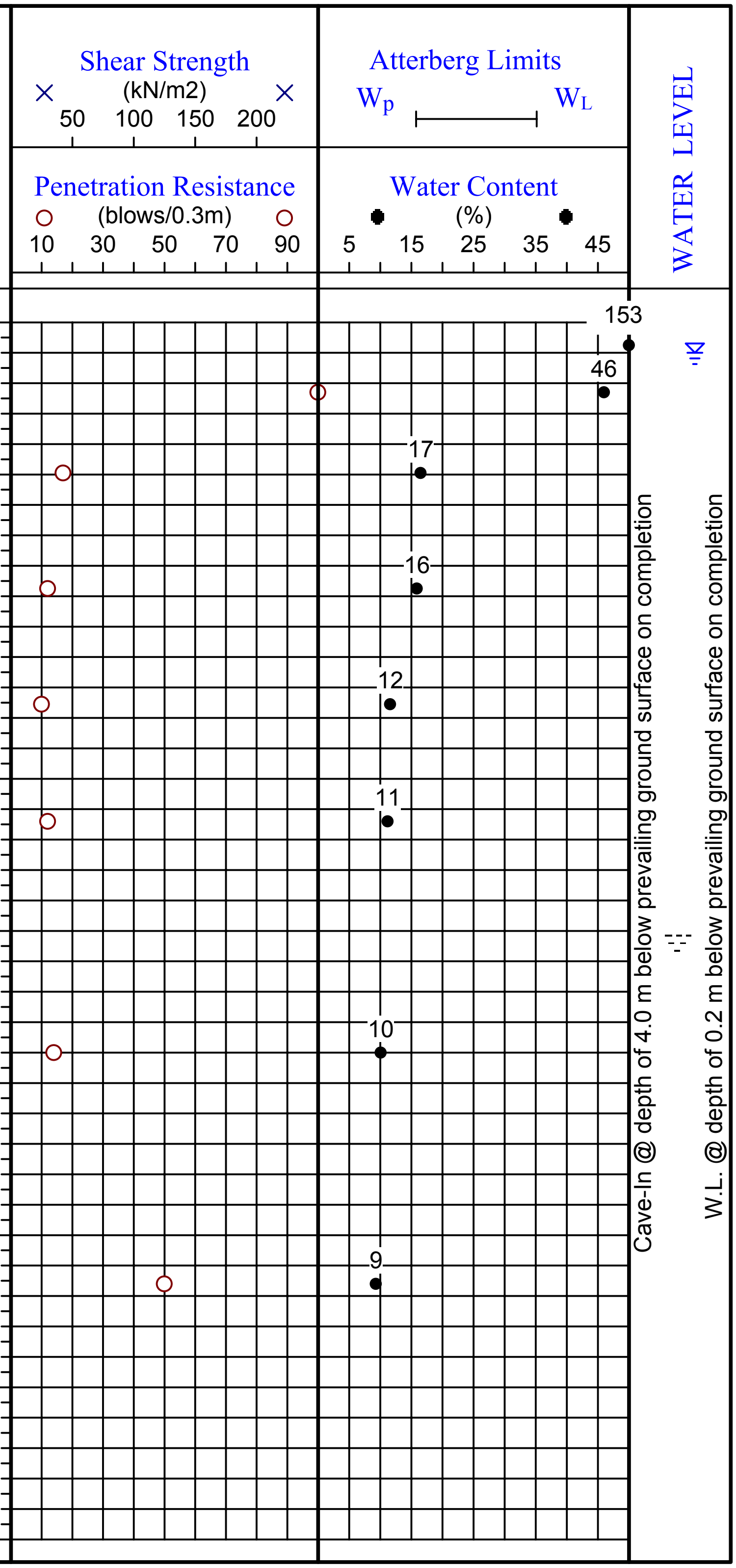
JOB DESCRIPTION: Proposed Residential Development

JOB LOCATION: 1657 Mapleview Road, Town of Innisfil

METHOD OF BORING: Flight-Auger

DATE: November 20, 2006

Elev. Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)
	Ground Surface				
0.0	**38cm TOPSOIL, PEATY**	1A	DO	-	0
	Stiff to hard; fine sand layer	1B	DO	100+	
	weathered	2	DO	17	1
	SILTY CLAY, TILL	3	DO	12	2
	sandy, a trace of gravel occ. sand and silt seams and layers, cobbles and boulders	4	DO	10	
		5	DO	12	3
	brown / grey				4
		6	DO	14	5
5.5	Grey, very dense **SANDY SILT, Till** traces of gravel and clay occ. cobbles, boulders, and wet sand and silt seams and layers	7	DO	50	6
6.6	**END OF BOREHOLE**				7, 8



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JOB NO.: 0611-S047

LOG OF BOREHOLE NO.: 4

FIGURE NO.: 4

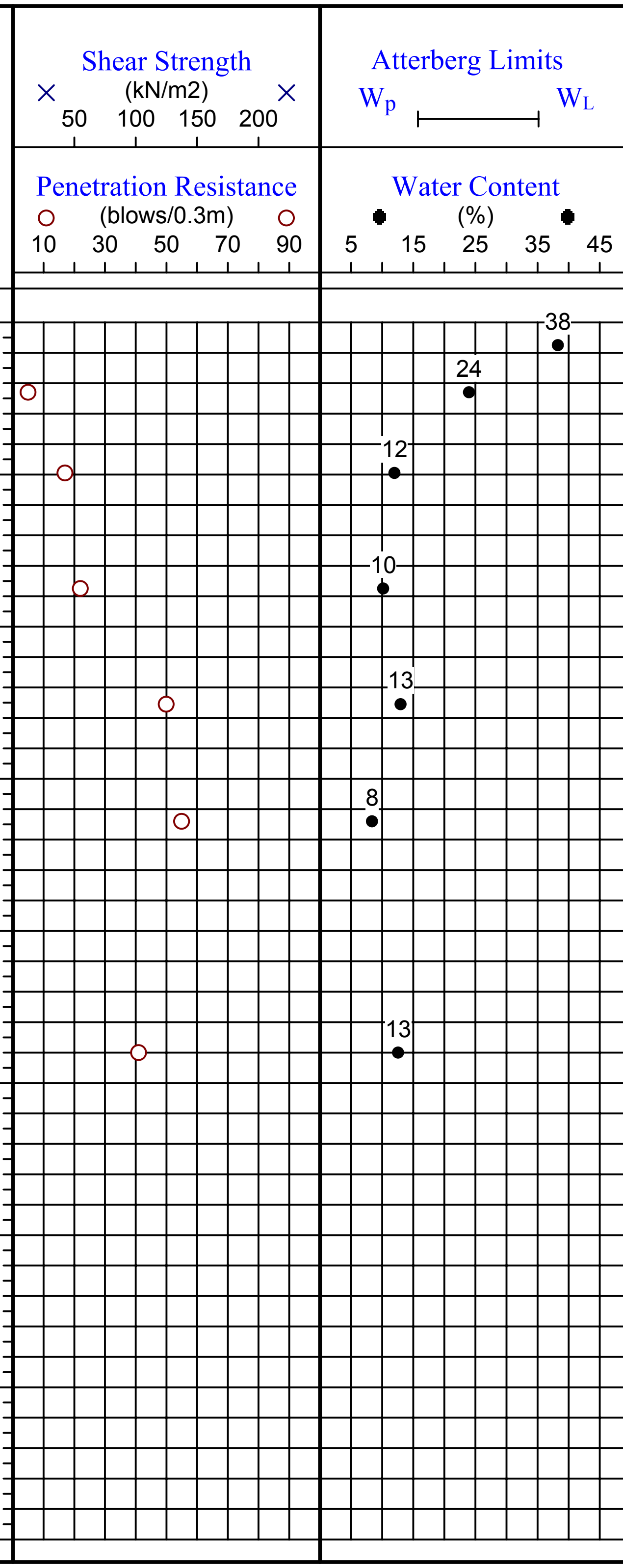
JOB DESCRIPTION: Proposed Residential Development

JOB LOCATION: 1657 Mapleview Road, Town of Innisfil

METHOD OF BORING: Flight-Auger

DATE: November 20, 2006

Elev. Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)
	Ground Surface				
0.0	33cm TOPSOIL	1A	DO	-	0
	Firm to hard	1B	DO	5	
	weathered	2	DO	17	1
	SILTY CLAY, Till	3	DO	22	2
	brown / grey	4	DO	50	
	sandy, a trace of gravel occ. sand and silt seams and layers, cobbles and boulders	5	DO	55	3
4.0	Grey, dense SANDY SILT, Till traces of gravel and clay occ. cobbles, boulders, and wet sand and silt seams and layers	6	DO	41	4
5.0	END OF BOREHOLE				5



WATER LEVEL

W.L. @ depth of 1.8 m below prevailing ground surface on completion

JOB NO.: 0611-S047

LOG OF BOREHOLE NO.: 5

FIGURE NO.: 5

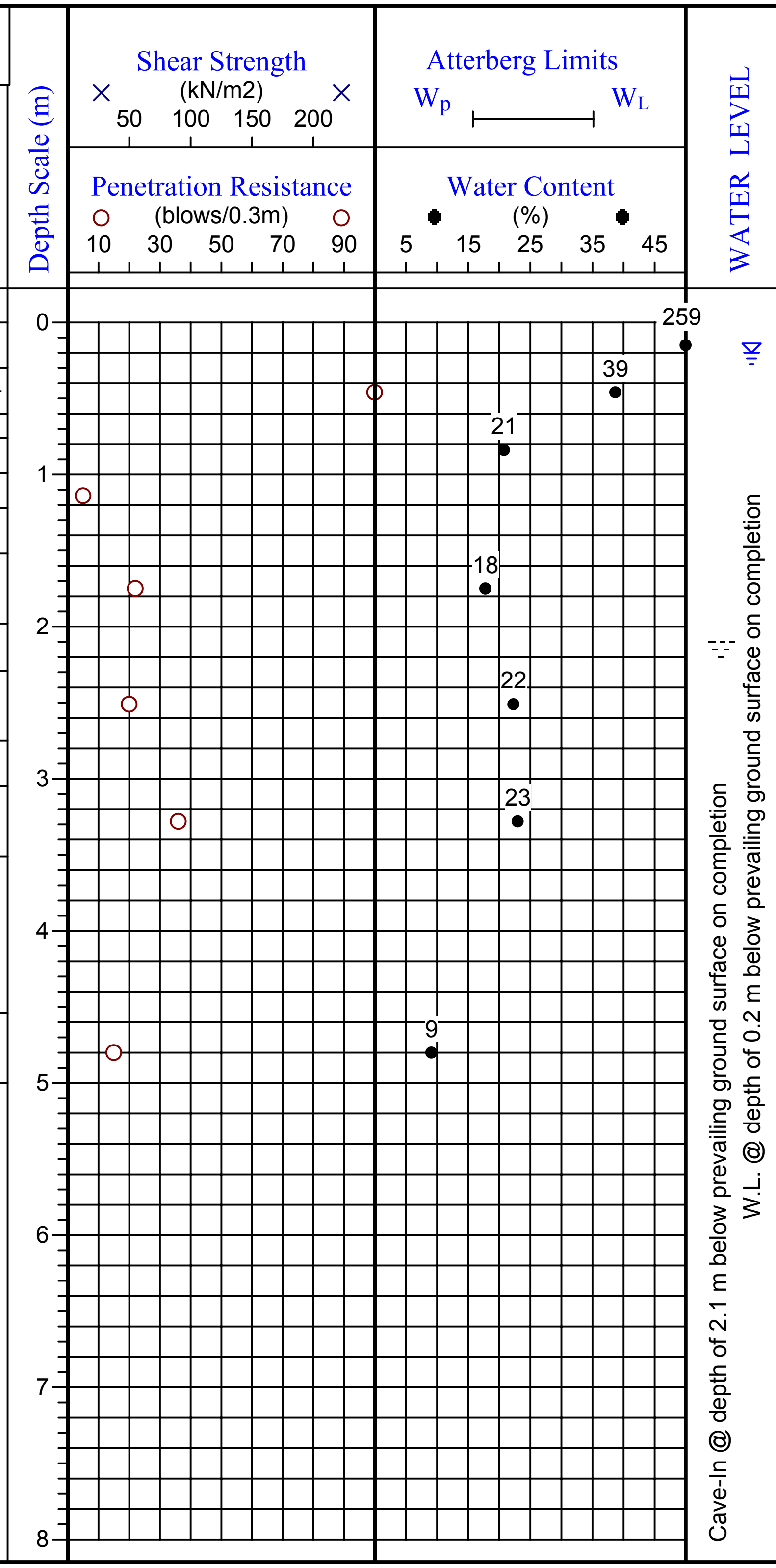
JOB DESCRIPTION: Proposed Residential Development

JOB LOCATION: 1657 Mapleview Road, Town of Innisfil

METHOD OF BORING: Flight-Auger

DATE: November 20, 2006

Elev. Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
	Ground Surface			
0.0	Dark brown, peaty **60cm TOPSOIL** with wood inclusions	1A	DO	-
		1B	DO	100+
	silty clay layer	2A	DO	-
	Brown, loose to compact **SILTY FINE SAND** occ. fine sand layers	2B	DO	5
		3	DO	22
2.1	Brown, very stiff to hard **SILTY CLAY** with wet silt and sand layers	4	DO	20
		5	DO	36
3.9	Grey, stiff **SILTY CLAY, Till** sandy, a trace of gravel occ. sand and silt seams and layers, cobbles and boulders	6	DO	15
5.0	**END OF BOREHOLE**			



JOB NO.: 0611-S047

LOG OF BOREHOLE NO.: 6

FIGURE NO.: 6

JOB DESCRIPTION: Proposed Residential Development

JOB LOCATION: 1657 Mapleview Road, Town of Innisfil

METHOD OF BORING: Flight-Auger

DATE: November 20, 2006

Elev. Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Penetration Resistance (blows/0.3m)	Water Content (%)	WATER LEVEL
	Ground Surface							
0.0	**30cm TOPSOIL**	1A	DO	-	0		39	
	SILTY CLAY, Till Firm to very stiff weathered	1B	DO	4		4	29	
		2	DO	20	1	20	10	
	brown	3	DO	20		20	9	W.L. @ depth of 1.8 m below prevailing ground surface on completion
	grey	4	DO	21	2	21	9	
	sandy, a trace of gravel occ. sand and silt seams and layers, cobbles and boulders	5	DO	19	3	19	10	
4.6	Grey, dense to very dense **SILTY SAND, Till** traces of gravel and clay occ. cobbles, bouldes and wet sand and silt seams and layers	6	DO	36	5	36	10	
		7	DO	100+	6	100+	9	
6.3	**END OF BOREHOLE**				7, 8			

Shear Strength (kN/m2): × 50 100 150 200 ×

Penetration Resistance ○ (blows/0.3m) ○: 10 30 50 70 90

Atterberg Limits: W_p ⊢—⊣ W_L

Water Content ● (%) ●: 5 15 25 35 45

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JOB NO.: 0611-S047

LOG OF BOREHOLE NO.: 7

FIGURE NO.: 7

JOB DESCRIPTION: Proposed Residential Development

JOB LOCATION: 1657 Mapleview Road, Town of Innisfil

METHOD OF BORING: Flight-Auger

DATE: November 20, 2006

Elev. Depth (m)	SOIL DESCRIPTION	Sample Number	Type	N-Value	Water Content (%)
	Ground Surface				
0.0	**30cm TOPSOIL**	1A	DO	-	43
	Brown, firm to hard	1B	DO	4	14
	weathered				
	SILTY CLAY, Till	2	DO	20	11
	sandy, a trace of gravel occ. sand and silt seams and layers, cobbles and boulders	3	DO	20	10
		4	DO	38	10
2.9	Compact to dense	5	DO	20	13
	SILTY SAND, Till				
	brown / grey				
	traces of gravel and clay occ. cobbles, boulders, and wet sand and silt seams and layers	6	DO	38	8
5.6	Grey, compact **SILTY FINE SAND** occ. fine sand layers	7	DO	25	18
6.6	**END OF BOREHOLE**				

Shear Strength (kN/m2): 50, 100, 150, 200

Penetration Resistance (blows/0.3m): 10, 30, 50, 70, 90

Atterberg Limits: W_p, W_L

Water Content (%): 5, 15, 25, 35, 45

Depth Scale (m): 0–8

WATER LEVEL

Cave-In @ depth of 3.1 m below prevailing ground surface on completion

W.L. @ depth of 2.7 m below prevailing ground surface on completion

Soil Engineers Ltd.

JOB NO.: 0611-S047

LOG OF BOREHOLE NO.: 8

FIGURE NO.: 8

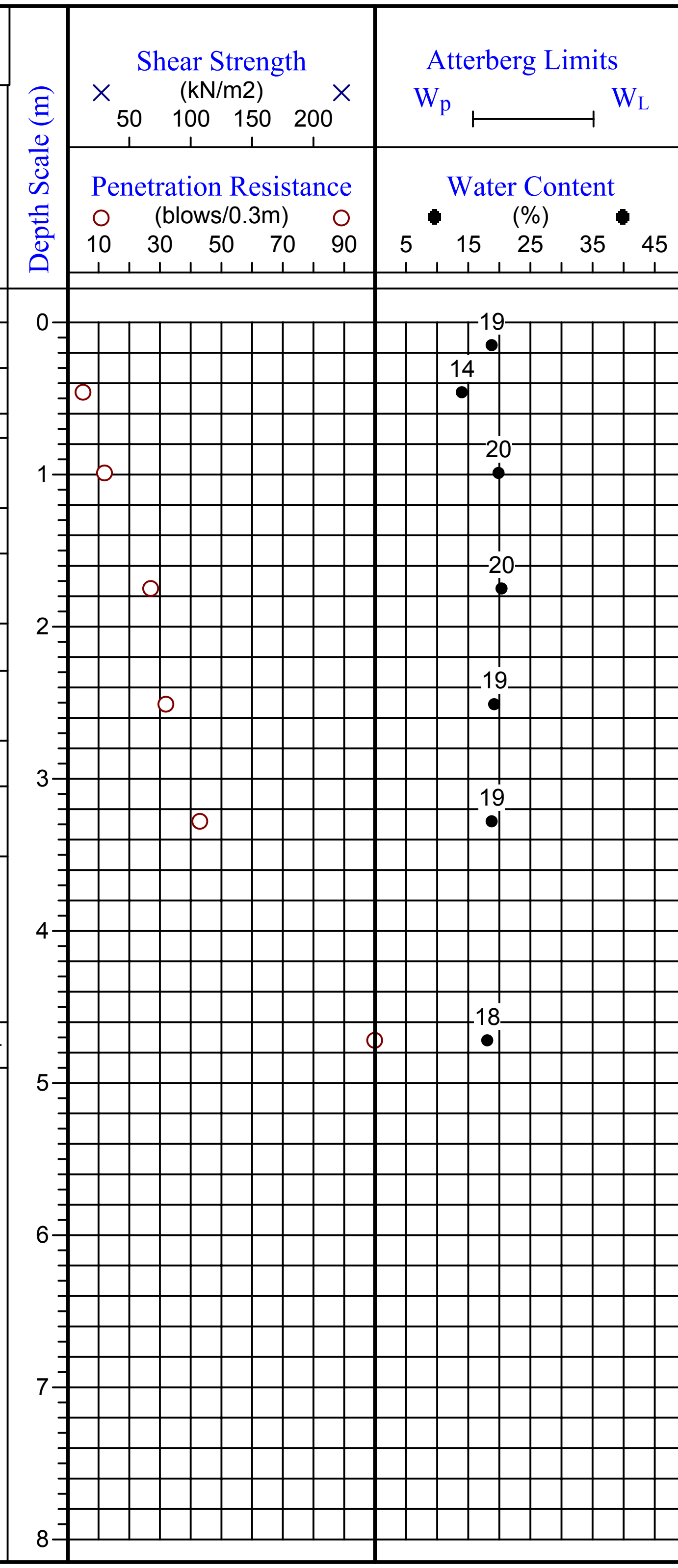
JOB DESCRIPTION: Proposed Residential Development

JOB LOCATION: 1657 Mapleview Road, Town of Innisfil

METHOD OF BORING: Flight-Auger

DATE: November 20, 2006

Elev. Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
	Ground Surface			
0.0	36cm TOPSOIL	1A	DO	-
	SILTY FINE SAND Brown, loose to very dense	1B	DO	5
	weathered	2	DO	12
		3	DO	27
		4	DO	32
		5	DO	43
	occ. fine sand layers	6	DO	100+
4.9	END OF BOREHOLE			



WATER LEVEL

Cave-In @ depth of 0.9 m below prevailing ground surface on completion

Soil Engineers Ltd.

JOB NO.: 0611-S047

LOG OF BOREHOLE NO.: 9

FIGURE NO.: 9

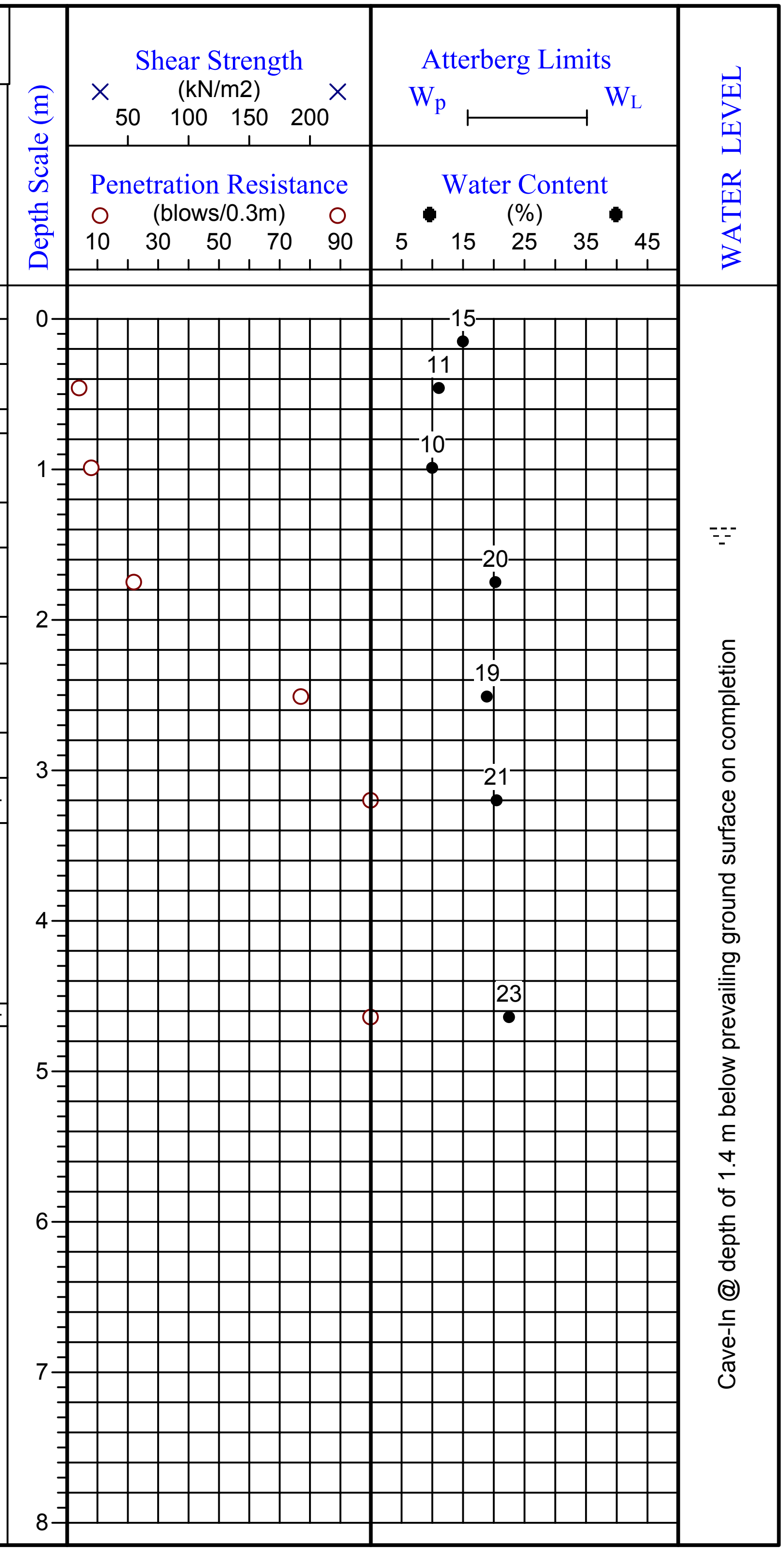
JOB DESCRIPTION: Proposed Residential Development

JOB LOCATION: 1657 Mapleview Road, Town of Innisfil

METHOD OF BORING: Flight-Auger

DATE: November 20, 2006

Elev. Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
	Ground Surface			
0.0	**28cm TOPSOIL**	1A	DO	-
	Brown, loose to very dense	1B	DO	4
	weathered	2	DO	8
		3	DO	22
	SILTY FINE SAND	4	DO	77
		5	DO	100+
	occ. fine sand layers	6	DO	100+
4.7	**END OF BOREHOLE**			



Soil Engineers Ltd.

JOB NO.: 0611-S047

LOG OF BOREHOLE NO.: 10

FIGURE NO.: 10

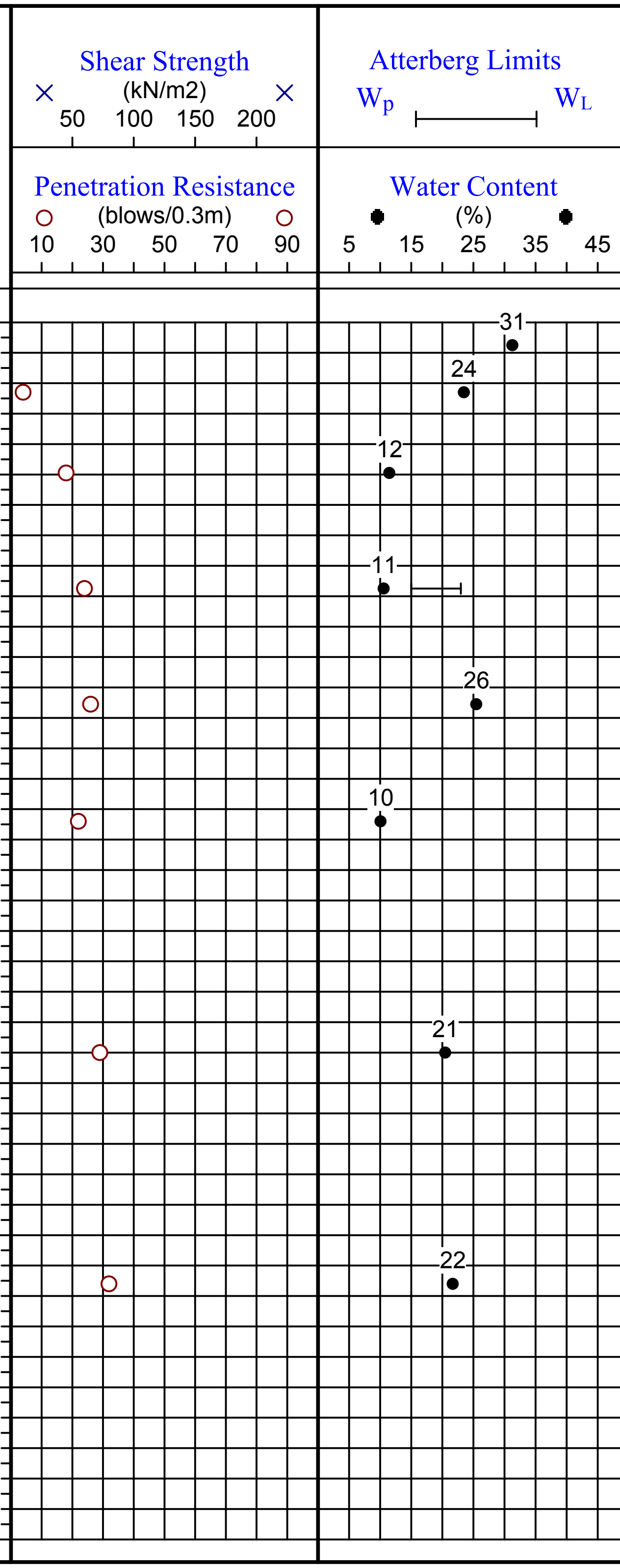
JOB DESCRIPTION: Proposed Residential Development

JOB LOCATION: 1657 Mapleview Road, Town of Innisfil

METHOD OF BORING: Flight-Auger

DATE: November 20, 2006

Elev. Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)
	Ground Surface				
0.0	**25cm TOPSOIL**	1A	DO	-	0
	Firm to very stiff	1B	DO	4	
	weathered				
		2	DO	18	1
	SILTY CLAY, Till	3	DO	24	
	brown				2
	grey wet sand layer	4	DO	26	
	sandy, a trace of gravel occ. sand and silt seams and layers, cobbles and boulders	5	DO	22	3
					4
4.3	Brown, compact to dense				
		6	DO	29	5
	SILTY FINE SAND				
	occ. fine sand layers				6
		7	DO	32	
6.6	**END OF BOREHOLE**				7
					8



WATER LEVEL

W.L. @ depth of 2.7 m below prevailing ground surface on completion

Cave-In @ depth of 4.3 m below prevailing ground surface on completion

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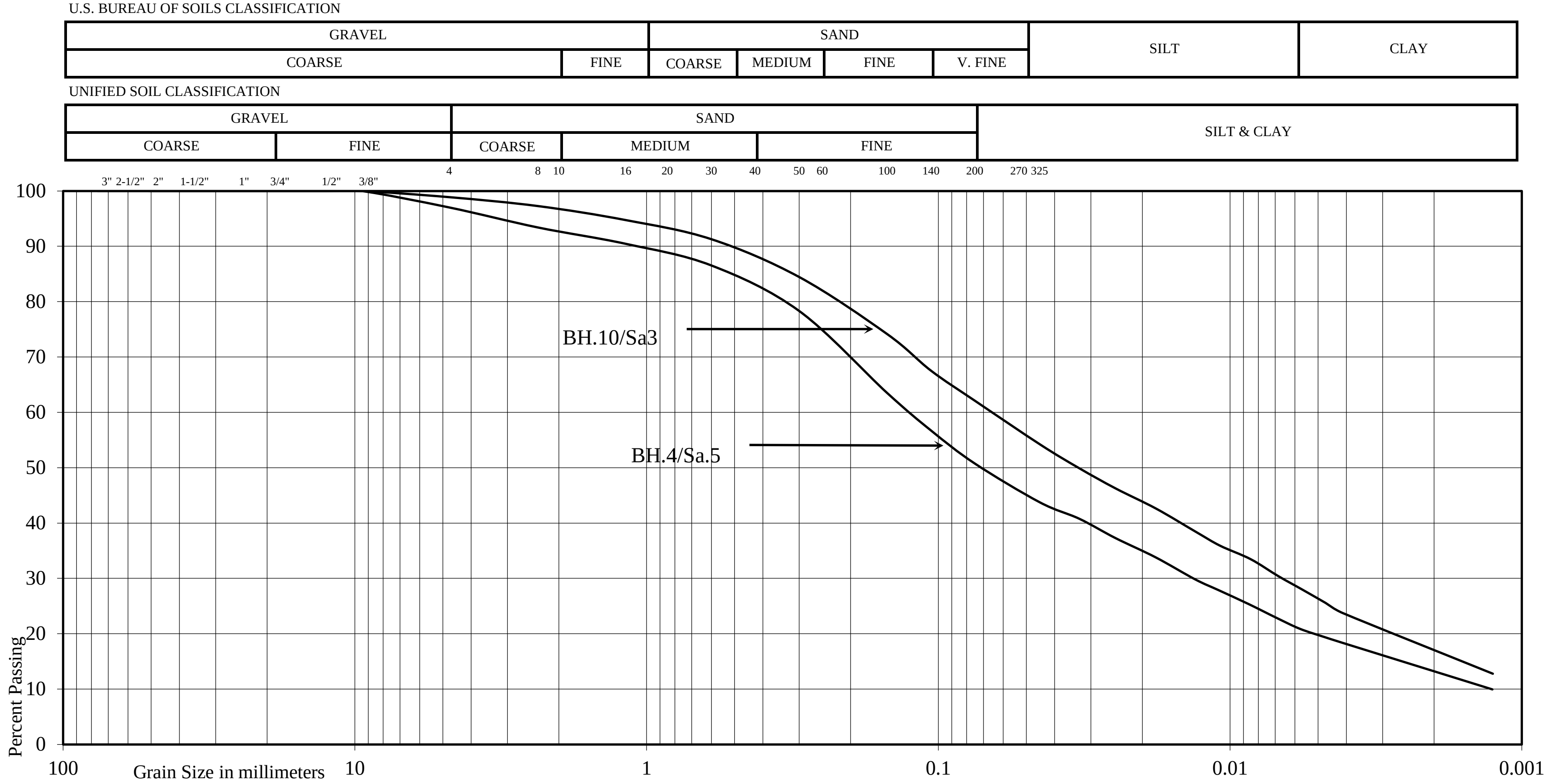
GRAIN SIZE DISTRIBUTION

Reference No: 0611-S047



Project:	Proposed Residential Development	
Location:	1657 Mapleview Road, Town of Innisfil	
Borehole No:	4	10
Sample No:	5	3
Depth (m):	3.3	1.8
Elevation (m):	-	-

BH./Sa.	4/5	10/3
Liquid Limit (%) =	-	23
Plastic Limit (%) =	-	15
Plasticity Index (%) =	-	8
Moisture Content (%) =	-	11
Estimated Permeability (cm./sec.) =	10^{-7}	10^{-7}

Classification of Sample [& Group Symbol]: SILTY CLAY, Till
sandy, a trace of gravel

Figure: 11

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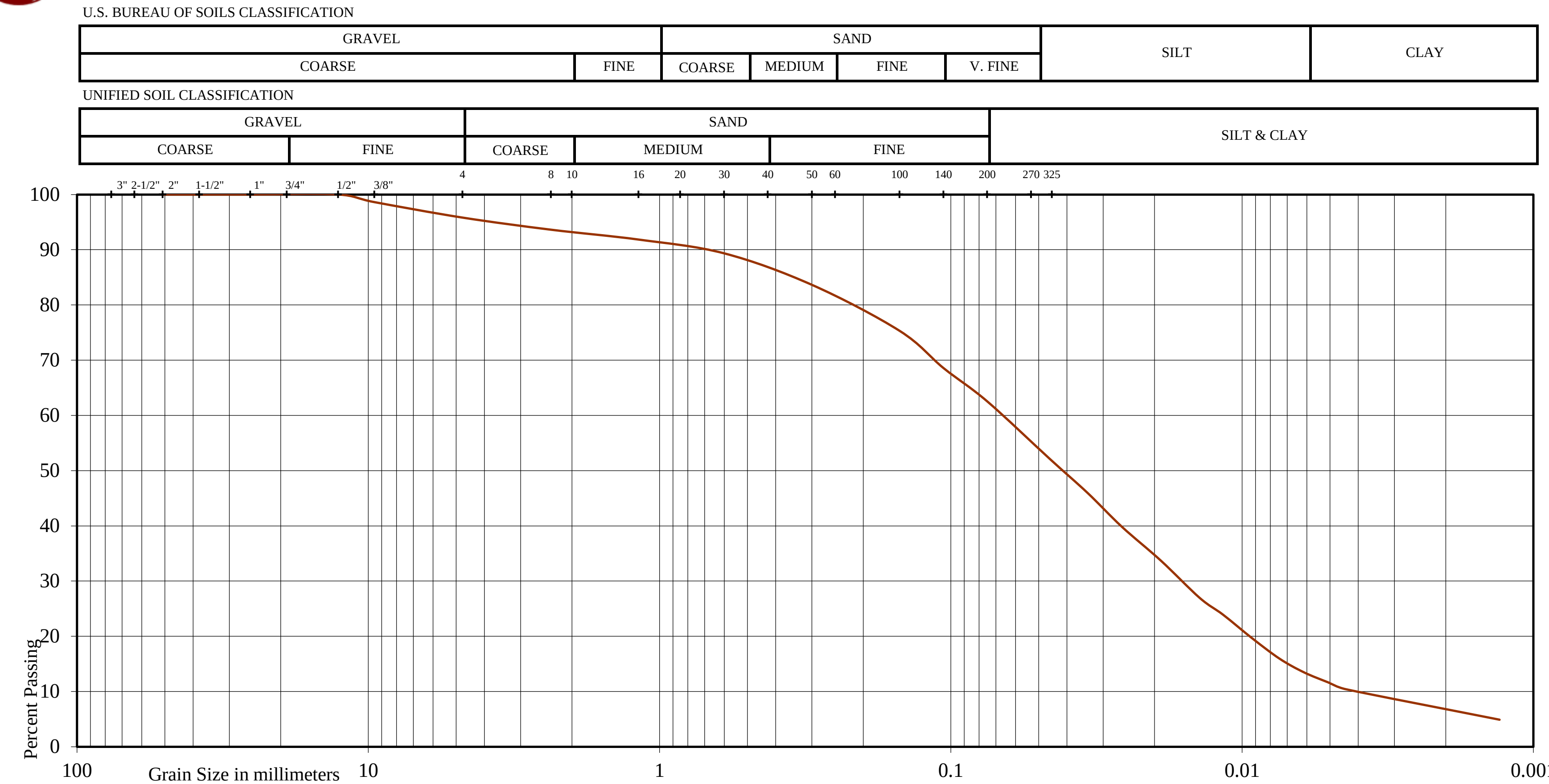
GRAIN SIZE DISTRIBUTION

Reference No: 0611-S047



Project: Proposed Residential Development

Location: 1657 Mapleview Road, Town of Innisfil

Borehole No: 4

Sample No: 6

Depth (m): 4.8

Elevation (m): -

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = -

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

Classification of Sample [& Group Symbol]:	SANDY SILT, Till traces of gravel and clay

Figure: 12

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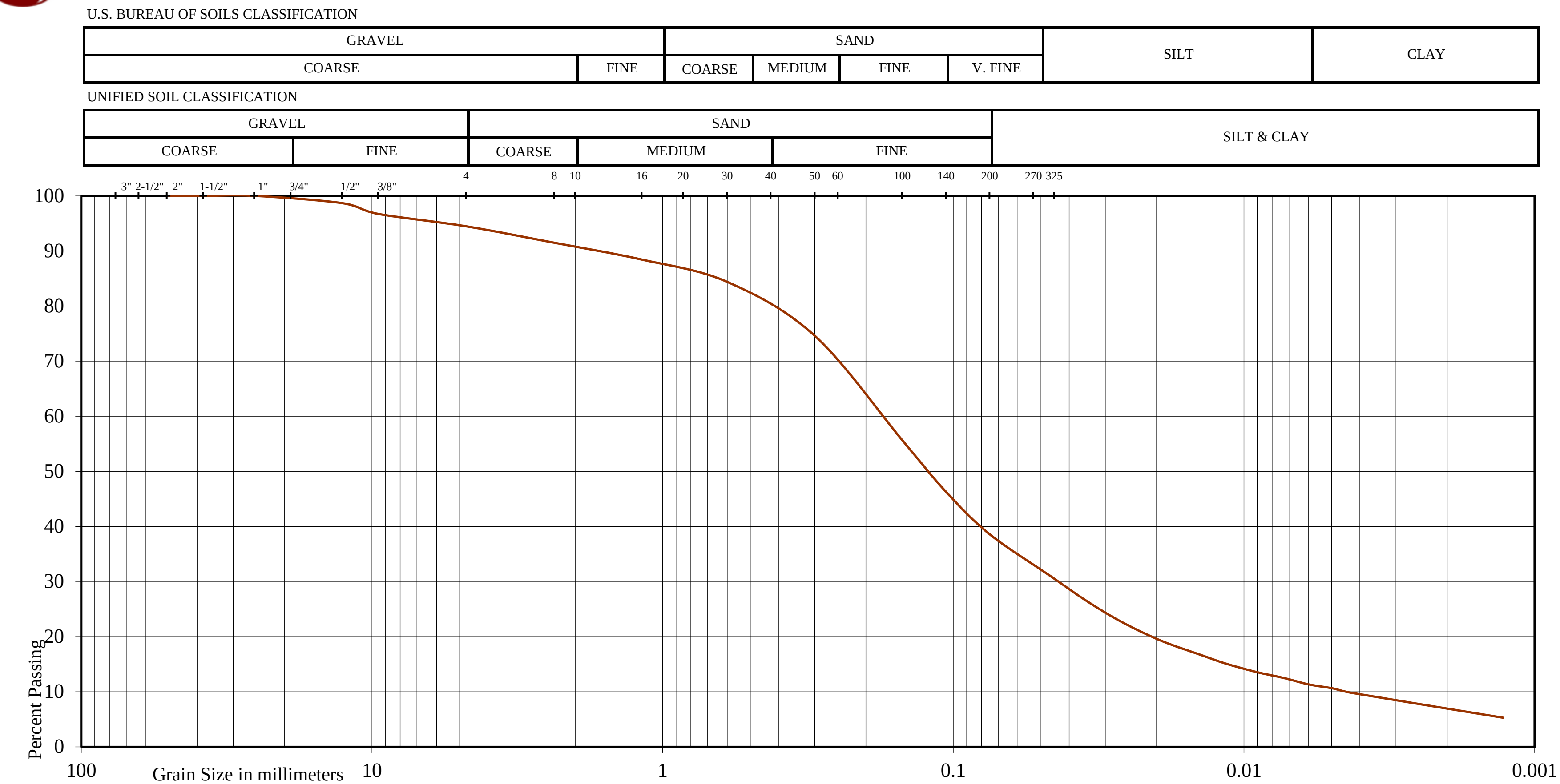
GRAIN SIZE DISTRIBUTION

Reference No: 0611-S047



Project:	Proposed Residential Development		
Location:	1657 Mapleview Road, Town of Innisfil	Liquid Limit (%) =	-
		Plastic Limit (%) =	-
Borehole No:	6	Plasticity Index (%) =	-
Sample No:	6	Moisture Content (%) =	-
Depth (m):	4.8	Estimated Permeability	
Elevation (m):	-	(cm./sec.) =	10^{-6}

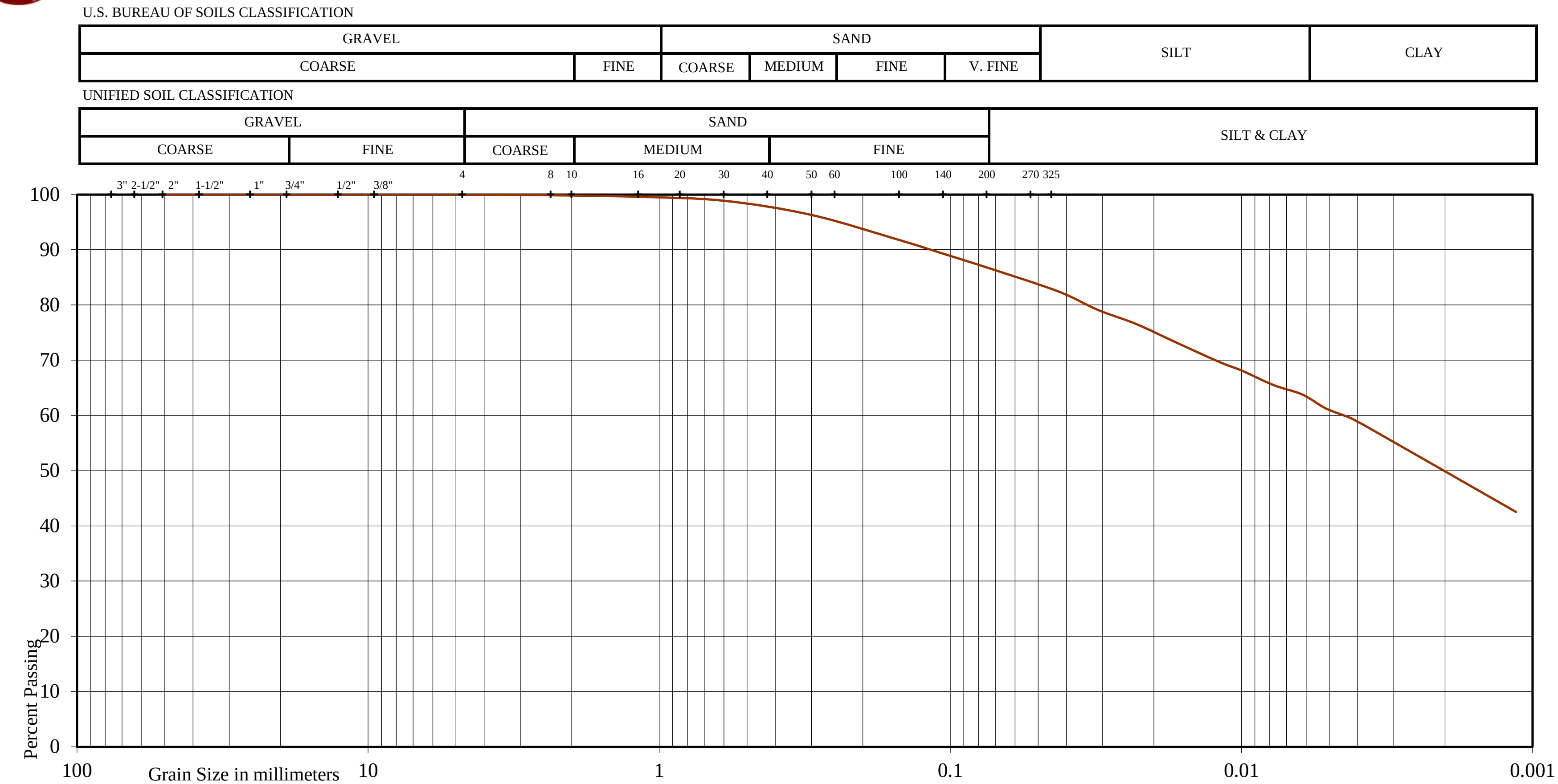
Classification of Sample [& Group Symbol]: SILTY SAND, Till
traces of gravel and clay

Figure: 13

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 0611-S047



Project:	Proposed Residential Development		
Location:	1657 Mapleview Road, Town of Innisfil	Liquid Limit (%) =	37
		Plastic Limit (%) =	20
Borehole No:	1	Plasticity Index (%) =	17
Sample No:	4	Moisture Content (%) =	29
Depth (m):	2.6	Estimated Permeability	
Elevation (m):	-	(cm./sec.) =	10^{-8}

Classification of Sample [& Group Symbol]: SILTY CLAY
some sand

Figure: 14

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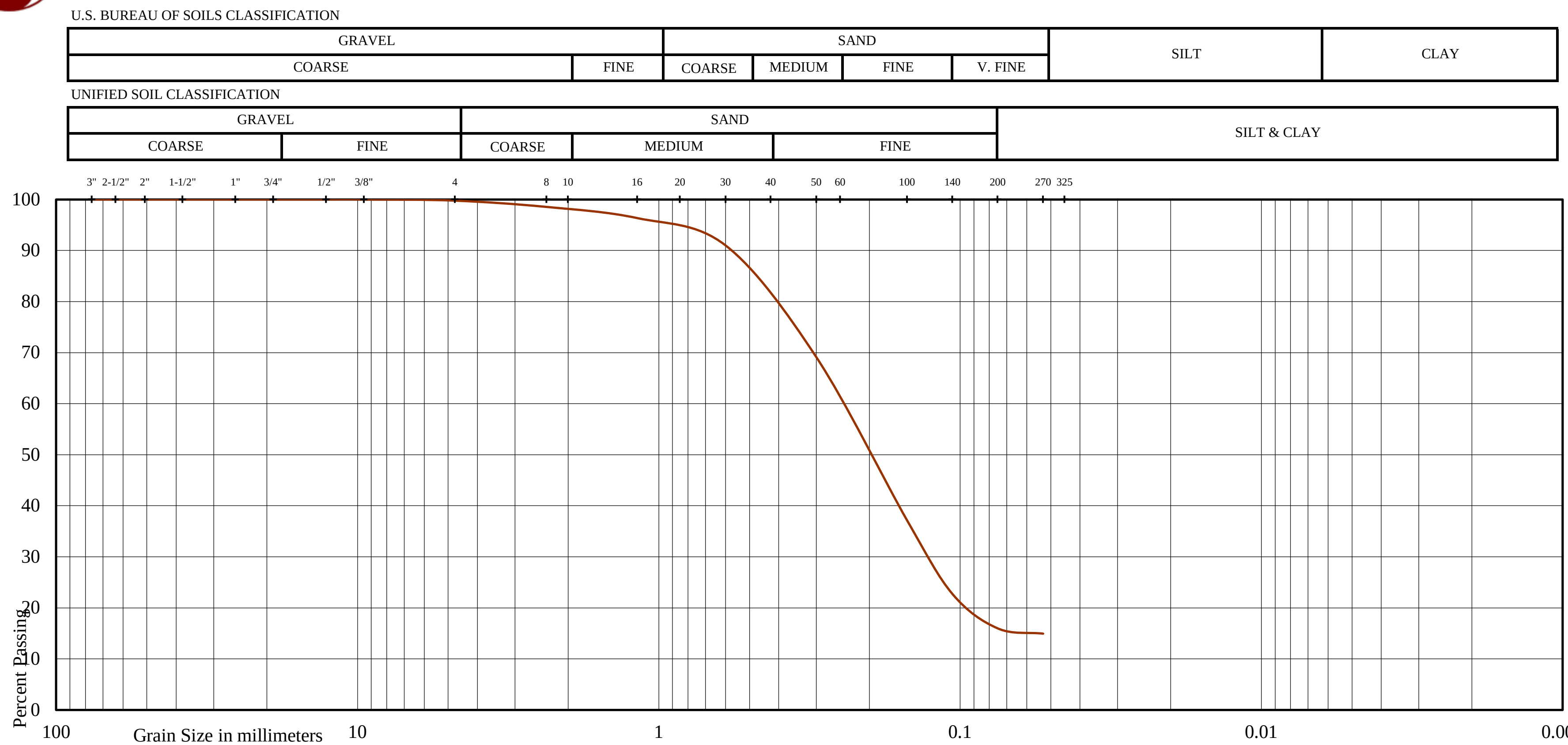
GRAIN SIZE DISTRIBUTION

Reference No: 0611-S047



Project: Proposed Residential Development

Location: 1657 Mapleview Road, Town of Innisfil

Borehole No: 1

Sample No: 2

Depth (m): 1.0

Elevation (m): -

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = -

Estimated Permeability (cm./sec.) = 10^{-3}

Classification of Sample [& Group Symbol]:	FINE TO MEDIUM SAND some silt, a trace of course sand

Figure: 15

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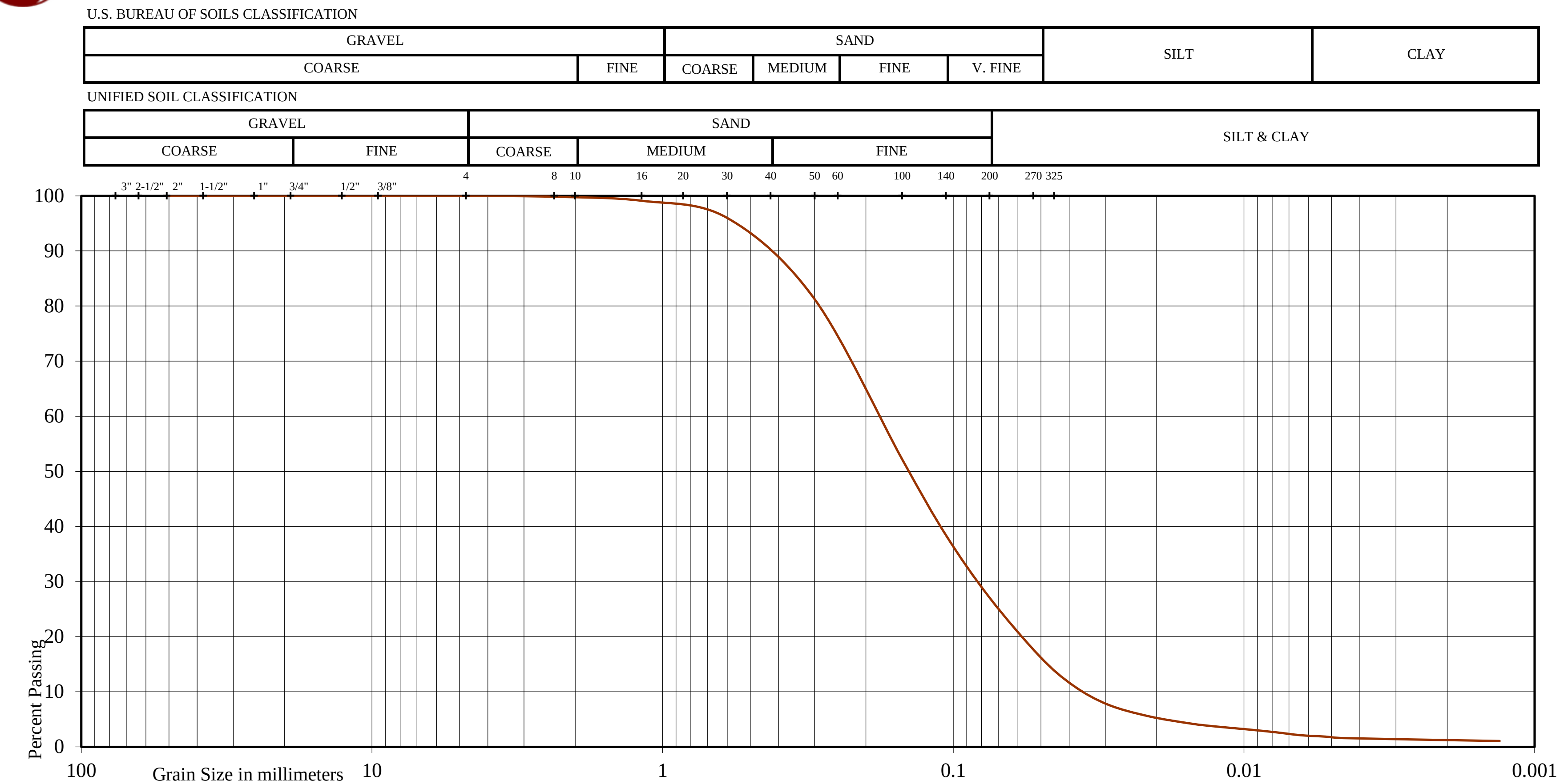
GRAIN SIZE DISTRIBUTION

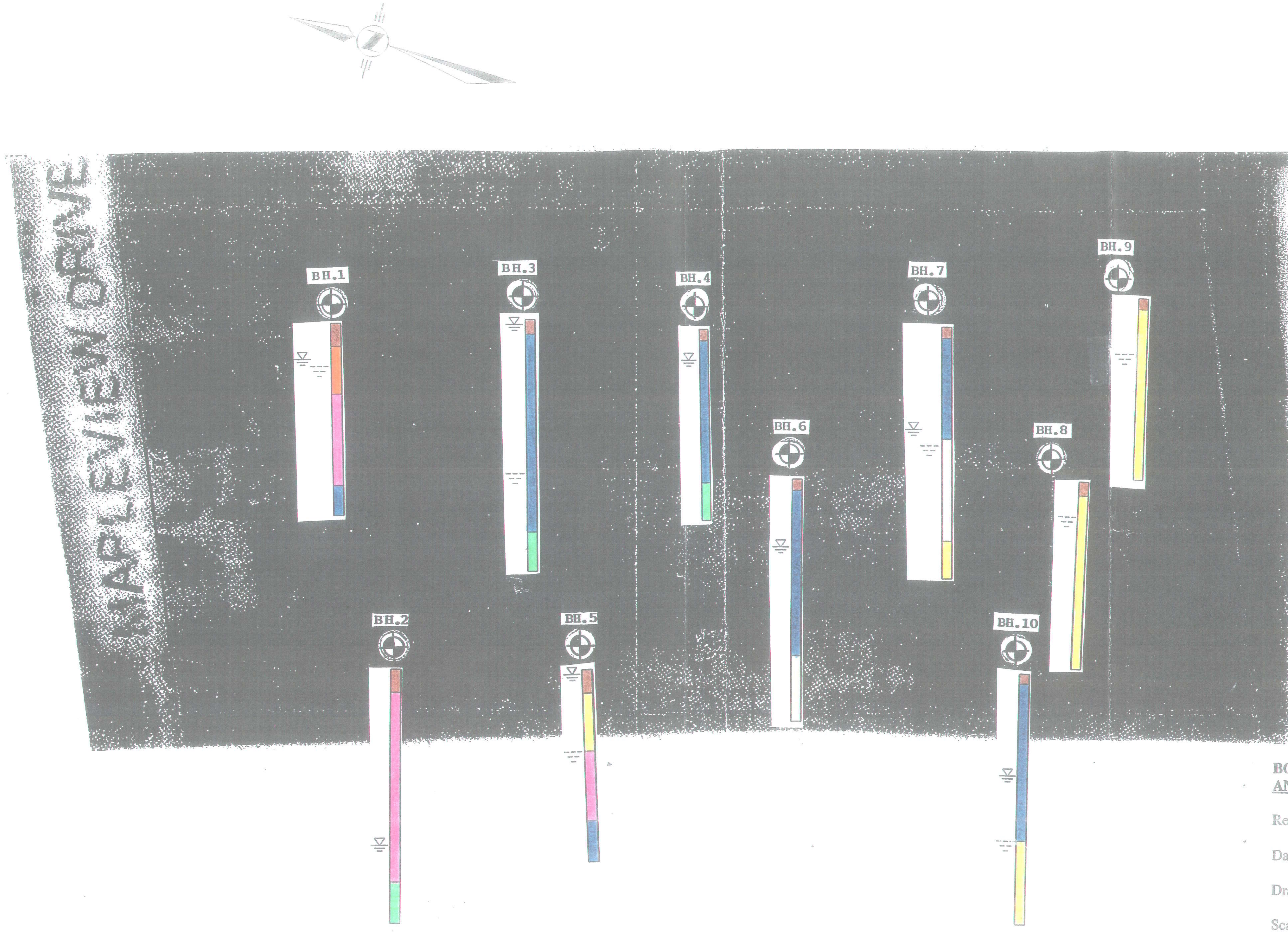
Reference No: 0611-S047



Project:	Proposed Residential Development		
Location:	1657 Mapleview Road, Town of Innisfil	Liquid Limit (%) =	-
		Plastic Limit (%) =	-
Borehole No:	8	Plasticity Index (%) =	-
Sample No:	4	Moisture Content (%) =	-
Depth (m):	2.6	Estimated Permeability	
Elevation (m):	-	(cm./sec.) =	10^{-4}

Classification of Sample [& Group Symbol]: SILTY FINE SAND

Figure: 16



BOREHOLE LOCATION PLAN AND SUBSURFACE PROFILE

Ref. No.: 0611-S047

Date: December 2006

Drawing No.: 1

Scale: Vert: 1:3000

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