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A REPORT TO
SHORELINE TOWERS INC.

A SOIL INVESTIGATION FOR PROPOSED CONDOMINIUM WITH 3-LEVEL UNDERGROUND PARKING

PART OF LOTS 377, 378 AND 379
BEHIND 2313 AND 2323 LAKE SHORE BOULEVARD WEST

CITY OF TORONTO

Reference No. 1203-S013

OCTOBER 2014
(Revision of Report dated May 2012)

DISTRIBUTION

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1.0 **INTRODUCTION**

In accordance with the written authorization dated March 1, 2012, from Mr. Stephen Greenberg, President, of Shoreline Towers Inc., a soil investigation was carried out at part of Lots 377, 378 and 379, behind 2313 and 2323 Lake Shore Boulevard West, in the City of Toronto, for a proposed Condominium with 3-Level Underground Parking.

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 on Iroquois Lake plain where drift has been partly eroded by the water action of the glacial lake and, in places, filled with lacustrine sand, silt, clay and reworked till. It beds onto shale bedrock of Georgian Bay Formation at moderate to considerable depths.

The subject site is irregular in shape and consists of a paved area used as a local parking. The ground is generally flat and level; the site is bordered by Lake Ontario easements to the east, existing buildings to the west, and landscaping/parking lots to the north and the south.

The proposed project consists of a condominium building with a 3 level underground parking/basement. The project will be provided with municipal services, access roadways and a loading area.

3.0 FIELD WORK

The field work, consisting of 5 boreholes to depths ranging from 6.9 to 11.3 m, was performed on March 28, 29 and 30, 2012, at the locations shown on the Borehole Location Plan, Drawing No. 1. It should be pointed out that some of the boreholes were relocated from their original proposed locations due to access difficulties and/or interference with the existing underground services.

The holes were advanced at intervals to the sampling depths by a truck-mounted, continuous-flight power-auger machine equipped for soil sampling. Standard Penetration Tests (SPT), 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.

‘NQ’ size rock coring was carried out in Boreholes 1 and 4 to assess the quality and soundness of the encountered shale bedrock. The quality of the rock has been assessed by applying the ‘Rock Quality Designation’ (RQD) classification, considering the total length of the recovered core pieces 10 cm or longer against the length of the core run. The results are expressed as a percentage and are recorded on the Borehole Logs.

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

The elevation at each borehole location was surveyed with a Global Navigation Satellite System (GNSS) with 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 5, inclusive. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing No. 2, and the engineering properties of the disclosed soils are discussed herein.

The investigation has disclosed that beneath an existing pavement structure and, in most locations, a layer of earth fill, the site is underlain by a stratum of silty clay. A localized deposit of silty fine sand was encountered in 1 borehole. Peat and alluvial deposits were encountered in most boreholes. The boreholes were terminated in shale bedrock which extended to the maximum investigated depths.

4.1 Pavement Structure (All Boreholes)

The existing pavement structure of the parking lot, as disclosed by the boreholes, is shown in Table 1.

Table 1 - Revealed Pavement Structure

BH No.	Asphaltic Concrete Thickness (mm)	Granular Fill Thickness (mm)	Subgrade Description
1	65	180	Earth Fill
2	50	200	Earth Fill
3	50	560	Silty Clay
4	40	125	Earth Fill
5	50	330	Earth Fill

The granular fill, in places, was mixed with the underlying fill by the infiltration of fines through cracks in the pavement and/or the upfiltration of the fill subgrade under traffic loads; this causes difficulty in delineating the interface between the earth fill and the granular fill.

The granular fill consists of crushed stone and well-graded sand, with a variable amount of silt, and appeared to be contaminated by the subgrade material.

Tactile examinations of the granular material indicate that it contains some silt (i.e., passing the Sieve No. 200).

The water content of the granular fill ranges from 4% to 10%, with a median of 5%, showing that the fill is generally in a damp condition.

Grain size analyses were performed on 3 representative samples of the granular fill; the results are plotted on Figure 6. The results show that the samples fail to meet the Gradation Requirements of the OPS Specifications for Granular 'A' or 'B', with excessive silt contents of 13% and 18%. This indicates that the granular fill is unsuitable for use as a road base material; however, if carefully salvaged, it can be used for structural backfill, road subgrade stabilization or bedding material, or as a granular sub-base material for construction of the shoulders.

Due to the variable silt content, the existing granular base material is moderately to highly frost susceptible.

4.2 **Earth Fill** (All Boreholes, except Borehole 3)

The earth fill mainly consists of silty clay material and extends to depths ranging from 1.2 to 4.9 m from the pavement surface. Sample examinations indicate that the fill contains occasional topsoil inclusions, brick fragments, foreign matter and some sand.

The lower layer of the fill in Boreholes 2 and 4 consists of mainly fine to coarse sand. In Borehole 2, hard resistance to SPT and augering at depths ranging from 3.8 to 4.9 m was encountered, showing rock fill occurred at this depth. The rock fill is considered to be localized.

The natural water content values of the samples were determined and the results are plotted on the Borehole Logs. The values range from 14% to 33%, with a median of 20%, showing the fill is in a moist to saturated, generally wet condition.

The obtained ‘N’ values range from 3 to 37, with a median of 8 blows per 30 cm of penetration. This indicates that the fill was placed randomly, but has well self-consolidated. Some of the high ‘N’ values are likely due to the presence of gravel and other debris in the fill.

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

Due to its non-uniform density, and the presence of topsoil inclusions and other deleterious material, the earth fill is considered incapable of supporting foundations. For new pavement construction, the earth fill should be further assessed by test pits and proof-rolling prior to the pavement construction.

As previously noted, the earth fill is amorphous in structure, indicating that it will be prone to collapse in steep excavations, particularly if it is in a wet condition. Its engineering properties are generally similar to those of the silty clay till, described in the following section.

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.

4.3 **Peat** (Boreholes 1 and 5)

The peat was encountered below the layer of earth fill at a depth of 2.3± m and extends to depths of 3.1± m and 3.4 m below the pavement surface. The peat is amorphous-granular in texture and contains fine fibrous decaying vegetation. It was formed by the progressive accumulation of incompletely decomposed plants in a wet environment. The peat is black in colour and emits a decaying smell.

The natural water content values of the samples taken are 95% and 323%, indicating that the peat is highly compressible. Since the peat is derived from vegetation, it will generate volatile gases under anaerobic conditions. Therefore, the peat is void of engineering value and needs to be stripped for the project construction.

4.4 **Alluvial Deposit** (Boreholes 1, 2 and 5)

The alluvial deposit, 1.2 m thick, was encountered at depths ranging from 2.3 to 6.1 m below the pavement surface. It consists of a mixture of organic silt and silty clay. The organic silt is fibrous and amorphous-granular in texture, and it contains remnants of plant debris that accumulated on the flood plain along the coast of Lake Ontario.

The natural water content was found to be 12% and 58%. The high water content of this organic soil indicates it is highly compressible and very low in shear strength. The organic material will likely generate volatile gases under anaerobic conditions. It is void of engineering value and can only be used for landscaping purposes.

The obtained ‘N’ value was 8, indicating that the deposit is considered to be loose or firm.

4.5 **Silty Fine Sand** (Borehole 1)

The silty fine sand deposit was found immediately beneath the earth fill. It extends to a depth of 2.3 m below the pavement surface. A sample examination shows that the silty fine sand is non-cohesive and generally in a saturated condition. The wet sample became dilatant when shaken by hand.

The natural water content of the silty fine sand sample was determined and the result is plotted on the Borehole Log. The value was 23%, indicating that the deposit is likely water bearing.

The obtained ‘N’ value was 6, indicating that the relative density of the sand is loose. The high water content and low ‘N’ value show the deposit has been loosened by the weathering process.

A grain size analysis was performed on 1 representative sample of the silty fine sand, and the result is plotted on Figure 8.

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

- High frost susceptibility with high soil-adfreezing potential.
- High water erodibility.
- 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

- A frictional soil, its shear strength is derived from internal friction and is density dependent. Due to its dilatancy, the shear strength of the wet soil 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.
- In cuts, the wet soil will slough readily, run with seepage and boil with a piezometric head of about 0.4 m.
- A fair material to support pavement, with an estimated California Bearing Ratio (CBR) value of 8%.
- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 5000 ohm·cm.

4.6 **Silty Clay** (All Boreholes)

The silty clay was found immediately below the pavement structure in Borehole 3 and extending to depths ranging from 5.0 to 7.6 m in the rest of the boreholes where it beds onto shale bedrock. It contains wet silt and fine sand layers and has a 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, in places, the consistency of the clay becomes slightly softer with depth, indicating that the clay has stiffened by desiccation.

The obtained 'N' values range from 7 to 30, with a median of 17, indicating the relative density of the clay is firm to very stiff, being generally very stiff.

The obtained 'N' values have a slight decreasing trend with depth, which is contrary to normal conditions, showing that the clay has been softened by water action.

The Atterberg Limits of 3 representative samples and the natural water content values of all of the samples were determined; the results are plotted on the Borehole Logs and summarized below:

Liquid Limit	26%, 28% and 31%
Plastic Limit	15%, 16% and 17%
Natural Water Content	13% to 30% (median 18%)

The values show that the silty clay is low in plasticity. The natural water content is at its plastic limit, which is consistent with the very stiff consistency as inferred from the 'N' values. This shows that the low 'N' values may have resulted from the invisible fine fissures in the clay.

Hard resistance was encountered during augering through the stratum, particularly in the lower zone of the stratum where appreciable shale fragments occurred. This renders it difficult to delineate the interface of the silty clay and the underlying shale bedrock; it is likely that the layer of clay is part of a badly weathered clay-shale reversion.

Grain size analyses were performed on 3 representative samples of the silty clay, and the results are plotted on Figure 10.

Based on 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 fine sand and silt layers, the lateral permeability is higher than the vertical permeability. The estimated coefficient of permeability is 10^{-7} 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 fine sand (seams and 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 excavation, the very stiff clay crust will be stable in a relatively steep cut for a short duration; however, as water seepage saturates the fine sand layers, the sides will slough, and sheet collapse may occur without warning.
- A very poor pavement-supportive material, with an estimated CBR value of 3% or less.
- Moderately high corrosivity to buried metal, with an estimated electrical resistivity of 3500 ohm·cm.

4.7 **Shale Bedrock** (All Boreholes)

The encountered shale is of the Georgian Bay Formation and is a laminated, sedimentary, moderately soft rock composed predominantly of clay material. It is interbedded with (about 20%) thin sandstone and limy shale bands.

The upper layers of the shale are often fissured as a result of the weathering process and/or overstressing by glaciation. In places, it contains hard clay inclusions, which are the result of a clay-shale reversion. The weathered condition often extends to about 2.0 or + m below the surface of the bedrock. Infiltrated precipitation and groundwater from the overburden soils will often permeate the fissures in the rock and, in places, will be under subterranean artesian pressure. However, because the shale is a clay rock, it is considered to be a material of low permeability and a poor aquifer, and the groundwater yield from the rock will be limited.

The recovery of NQ rock cores drilled in Boreholes 1 and 4 was 94% to 100%, being generally over 95%; however, examination of the RQD shows 0% to 49%, with a median of 18%. This indicates that the shale quality within the core depth is considered to be very poor to poor.

Two uniaxial compressive strength tests were attempted on 2 core samples; however, the samples easily crumbled. The shale bedrock generally ranges from 7 to 40 MPa. Based on the RQD rating, the compressive strength of the shale is estimated to be about 10 MPa.

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 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.

When excavating the sound shale, 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.

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.

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 2.

Table 2 - Estimated Water Content for Compaction

Soil Type	Determined Natural Water Content (%)	Water Content (%) for Standard Proctor Compaction	
		100% (optimum)	Range for 95% or +
Granular Fill	4 to 10 (median 5)	7	4 to 12
Earth Fill	14 to 33 (median 19)	18	14 to 23
Silty Fine Sand	23	11	5 to 16
Silty Clay	13 to 30 (median 18)	17	13 to 22
Shale	5 to 13 (median 9)	10	7 to 17

Based on the above values, the silty clay is generally suitable for 95% or + Standard Proctor compaction. However, portions of the silty clay and the silty fine sand are too wet for 95% or + Standard Proctor compaction and must be aerated prior to backfilling.

The clay should be compacted using a heavy-duty, kneading-type roller. The sand can be compacted by a smooth drum 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 very stiff 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 this soil 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; constant wetting of the sound clay which is generally on the dry side of optimum will be necessary to achieve this requirement.

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

As note4d, 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. If shale spoil is to be used immediately for structural backfill, it must be pulverized to sizes of 15 cm or less, must be compacted with lifts less than 15 cm, and will require continuous wetting during compaction. It should be compacted to achieve at least 95% of its maximum Standard Proctor dry density. The structurally compacted shale fragment fill must be left for a period of at least 1 winter to allow the shale to swell prior to the project construction.

5.0 **GROUNDWATER CONDITIONS**

Groundwater seepage encountered during augering was recorded on the field logs. The level of groundwater or occurrence of cave-in was measured upon completion of the boreholes; the data are plotted on the Borehole Logs and listed in Table 3.

Table 3 - Groundwater Levels

	Borehole	Soil Colour Changes Brown to Grey	Seepage Encountered During Augering		Measured Groundwater/ Cave-in* Level On Completion	
BH No.	**Depth (m)**	**Depth (m)**	**Depth (m)**	**Amount**	**Depth (m)**	**El. (m)**
1	9.8	4.6	1.5	Appreciable	1.6	74.7
2	7.1	6.1	2.4	Appreciable	1.8	74.9
3	6.9	3.2	-	-	Dry	-
4	11.3	6.6	2.4	Some	3.1	73.6
5	8.5	3.1	1.8	Some	3.4	73.6

The soil colour changes from brown to grey at depths ranging from 1.2 to 6.6 m below the pavement surface, indicating that the soils in the upper zone have oxidized. The groundwater regime lies within the grey-coloured soils or shale; it will fluctuate with the seasons and will most likely be affected by the steady water level in Lake Ontario.

The yield of groundwater from the silty clay, due to its low permeability, will be small and limited. However, the yield in the sand is expected to be moderate to appreciable, depending on its extent and continuity. It is known that the shale may contain occasional pockets of groundwater trapped in the rock fissures which may sometimes be under moderate subterranean artesian pressure. Upon release through excavation, this water will likely drain readily with limited yield.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation has disclosed that beneath a pavement structure and layers of earth fill extending to depths ranging from 0.6 to 4.9 m overlying peat and alluvial deposits, the site is underlain by a predominant stratum of firm to very stiff, generally very stiff silty clay overlying shale bedrock at depths ranging from 5.3± to 7.6 m below the pavement surface. The silty clay stratum is embedded with a localized layer of loose silty fine sand in 1 borehole, with sand and peat seams and layers at various locations and depths and will likely be affected by the steady water level in Lake Ontario.

Groundwater was measured in 4 of 5 boreholes on completion of the field work at depths ranging from 1.4 to 3.4 m. Groundwater seepage was detected in the upper layer showing the stabilized groundwater will likely rise to the seepage level. The groundwater will fluctuate with the seasons and will likely be affected by the steady water level in Lake Ontario.

The yield of groundwater from the silty clay, due to its low permeability, will be small and limited. However, the yield in the sand is expected to be moderate to appreciable, depending on its extent and continuity. The yield of groundwater from the shale bedrock, if any, may be appreciable initially but will be spent if drained continuously.

The geotechnical findings which warrant special consideration are presented below:

1. Due to the presence of the earth fill, peat seams and weathered shale, the footing subgrade must be inspected by a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to assess if the footing subgrade is compatible with the designed foundations.

2. The earth fill, peat and alluvial deposit are not suitable to support a structure; the earth fill must be subexcavated, sorted free of topsoil inclusions and deleterious materials, aerated and properly recompacted prior to being used for structural fill.
3. The sound natural soils and shale are suitable for normal spread and strip footing; where a higher bearing capacity is required, the foundation must extend onto the shale bedrock.
4. Due to the presence of adjacent buildings, the foundation details of the adjacent structures must be investigated and incorporated into the design and construction of the proposed project. It is recommended that a pre-construction survey and monitoring program be carried out for all adjacent structures in order to verify any potential future liability claims.
5. Large shale fragments, rock slabs and boulders over 15 cm in size are unsuitable for use as structural backfill and must be wasted.
6. The foundation of the proposed condominium, with 3-level underground parking, is expected to be 9.0 to 10.0 m below the pavement surface, or at the elevation of 67.0 to 68.0 m. Sound shale is anticipated at this level.
7. The sides of excavation in the overburden must be properly shored for stability and safety.
8. In general, open-cut excavation can be carried out by using a backhoe equipped with a rock-ripper up to 6.0 to 7.0 m below the prevailing ground surface; however, where deep trench excavations are required at 8.0+ m below the prevailing ground surface, particularly for an anticipated 3-level underground parking, pneumatic hammering with chisel points and/or rock blasting to break up the shale may be necessary for efficient rock removal.
9. Where underground services are to be placed into shale bedrock, the trench sides should be slightly sloped rather than vertical, due to the residual stress relief and the swelling characteristics of the shale. The side slopes should be

about 2 vertical:1 horizontal and lined with a cushioning layer such as compressible Styrofoam.

10. The peat and very soft clay should be subexcavated and replaced by properly compacted inorganic earth fill or granular material. All the granular bases should be compacted to their maximum Standard Proctor dry density.

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 Underground Garage

In general, Maximum Allowable Soil Pressures (SLS) of 200 kPa and 1000 kPa with corresponding Factored Ultimate Soil Bearing Pressures (ULS) of 350 kPa and 1600 kPa, respectively, can be used for the design of normal spread and strip footings and/or raft foundations founded on sound natural soils or shale bedrock. The suitable founding levels are presented in Table 4.

Table 4 - Founding Levels

BH No.	Maximum Allowable Soil Pressure (SLS)/ Factored Ultimate Soil Bearing Pressure (ULS) and Corresponding Founding Level					
	200 kPa (SLS) 350 kPa (ULS)		1000 kPa (SLS) 1600 kPa (ULS)		2500 kPa (ULS)*	
	Depth (m)	El. (m)	Depth (m)	El. (m)	Depth (m)	El. (m)
1	4.8 or +	71.5 or -	6.0 or +	70.3 or -	6.5 or +	69.8 or -
2	6.3 or +	70.4 or -	7.0 or +	69.7 or -	-	-
3	1.8 or +	74.8 or -	6.5 or +	70.1 or -	-	-
4	4.0 or +	72.7 or -	7.0 or +	69.7 or -	8.6 or +	68.1 or -
5	4.8 or +	71.8 or -	8.2 or +	68.4 or -	-	-

*The foundation will bed into the weathered shale. The settlement of the foundation on rock is generally impacted by the Factored Ultimate Soil Bearing Pressure of the rock.

It is understood that the proposed condominium building will contain a 3-level underground parking/basement. Based on the borehole findings, the subgrade at the main founding level will extend to depths ranging from 9.0 to 10.0 m below the pavement surface into the weathered or, likely, reasonably sound shale bedrock. However, the garage entrance ramp should be founded beneath the earth fill and the sound silty clay.

The reasonably sound shale generally occurs about 1.0 to 2.0 m from the shale surface. In areas where caissons are to be used, the ratio of the embedded soil depth to the diameter of the caisson should be at least 2:1. The centre-to-centre spacing between the caissons must be at least twice the diameter of the largest adjacent caisson base. The caisson excavation must be temporarily lined prior to concreting to prevent loose rocks from falling into the excavation. In order to facilitate inspections and cleaning of the founding subgrade, the size of the caisson should be at least 80 cm in diameter. Where caissons are ratcheted with the shale, a side adhesion of 1700 kPa can be used for calculating the bearing capacity of the caissons.

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

The foundation subgrade must be inspected by a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to ensure that the condition of the subgrade is compatible with the foundation design requirements.

Foundations exposed to weathering or adjacent to fresh air ducts in unheated areas must be protected against frost action by a minimum earth cover of 1.2 m. Perimeter wall and interior wall/column footings within an unheated garage should be founded at minimum depths of 0.6 m and 0.9 m, respectively, below the slab-on-grade elevation. If excavation into the shale is to be carried out close to the foundation walls, the sides of excavation into sound shale should be shielded by compressible Styrofoam (or equivalent). This will provide a cushioning layer against movement of the shale that may damage the basement walls.

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

Due to the presence of the adjacent buildings, the foundation details of the adjacent structures must be investigated and incorporated into the design and construction of the proposed project. It is recommended that a pre-construction survey and monitoring program be carried out for all adjacent structures in order to verify any potential future liability claims.

As required by the City of Toronto by-law, a vibration monitoring program will be required during construction and excavation into bedrock.

6.2 Underground Garage and Slab-On-Grade

For the underground garage, the perimeter garage walls should be designed to sustain a lateral earth pressure calculated using the soil/rock parameters stated in Section 6.7 and any applicable surcharge loads. Surcharge loads from adjacent structures must also be incorporated into the project design and construction.

The subgrade for the slab-on-grade of the underground garage/basement will consist of reasonably sound shale bedrock. A Modulus of Subgrade Reaction of 50 MPa/m can be used for the design of the floor slab founded on shale. Where the subgrade for the slab-on-grade consists of very stiff clay, a Modulus of Subgrade Reaction of 30 MPa/m can be used for the design of the floor slab. The floor slab-on-grade should be constructed on a granular base 20 cm thick, consisting of 20-mm Crusher-Run Limestone, or equivalent, compacted to its maximum Standard Proctor dry density.

Perimeter subdrains encased in a fabric filter will be required. The perimeter underground garage walls should be dampproofed and provided with synthetic sheet drains. As noted, if groundwater seepage is encountered in the bedrock, the groundwater yield is expected to be small and limited; however, in order to drain the accumulation of groundwater, a subdrain system consisting of 100-mm filter-sleeved weepers with 5.0 to 10.0 m spacing, depending on the amount of groundwater, must be installed below the granular base and must be connected to foundation drains or sump-wells. A vapour barrier must be provided at the crown level of the floor subdrains to prevent the emission of excessive water vapour. The ground around the building must be graded to direct water away from the structure to minimize the frost heave phenomenon generally associated with the disclosed soils. The requirements for the above measures can be further assessed during construction.

At the garage entrances, the subgrade should be properly insulated, or the subgrade material should be replaced with 1.0 m of non-frost-susceptible granular material and should be provided with subdrains. This will minimize frost action in this area where vertical ground movement cannot be tolerated. The floor at the entrances and in areas of close proximity to air shafts should be insulated, and the insulation should extend 5.0 m internally. This measure is to prevent frost action induced by cold wintry drafts.

6.3 Underground Services

The subgrade for the underground services should consist of sound natural soils, bedrock or uniformly compacted organic-free earth fill. 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, to be approved by a geotechnical engineer.

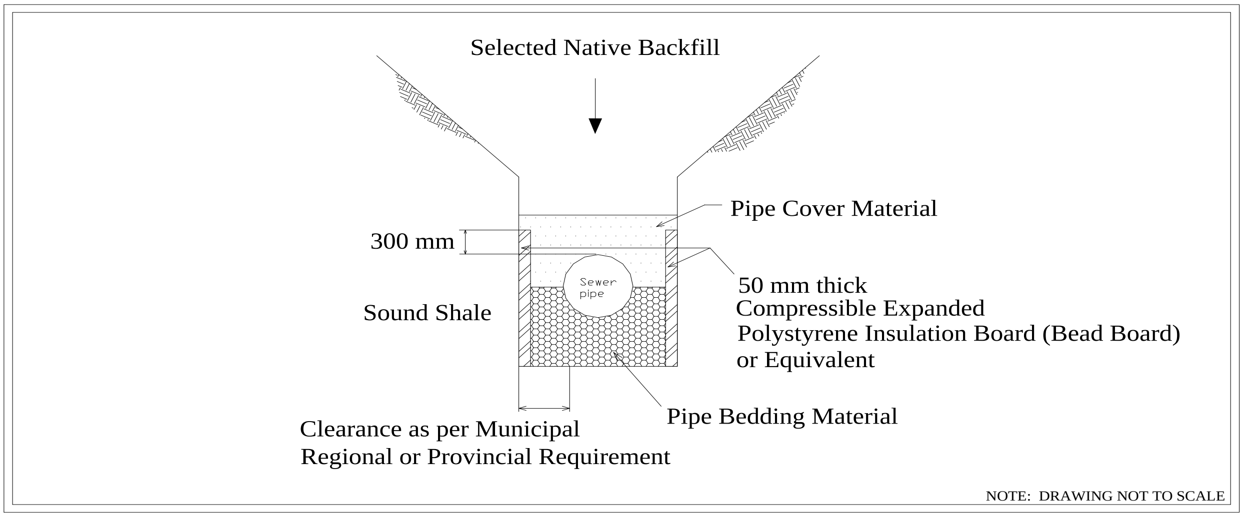
In order to prevent pipe floatation when the sewer trench is deluged with water, a soil cover with a thickness equal to the outside diameter of the pipe should be in place at all times after completion of the pipe installation.

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

Sewer construction will require rock excavation. In general, it can be carried out by using a backhoe equipped with a rock-ripper, but where trench excavation into the thick limy or sound shale is required, a pneumatic hammer should be used to break up the rock mass for excavation.

Where the pipe is to be placed into the sound shale bedrock, the trench sides should be slightly sloped rather than vertical, due to the residual stress relief and the swelling characteristics of the shale. The side slopes should be about 2 vertical: 1 horizontal. The rock face can be lined with a cushioning layer such as Styrofoam, then backfilled with sand up to 0.3 m above the crown of the pipe and flooded. The recommended scheme is illustrated in Diagram 1.

Diagram 1 - Sewer Installation in Sound Shale



As a general guide, an electrical resistivity of 3500 ohm·cm for silty clay and shale bedrock should be used to determine the mode of protection for the water main against soil corrosion. This should be confirmed by testing the soils along the water main alignment at the time of construction.

6.4 **Backfilling in Trenches and Excavated Areas**

The on-site inorganic soils are suitable for trench backfill. However, the clay should be sorted free of large pieces (over 15 cm in size) of limy shale, sandstone and shale fragments, or the large pieces must be broken into sizes suitable for structural compaction.

The lifts of each backfill layer should be limited to a thickness of 20 cm, or to a suitable thickness as determined by test strips to be carried out at the time of compaction.

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 achievement of the proper compaction. In confined areas where the desired slope cannot be achieved or the operation of a proper kneading-type roller cannot be facilitated, 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 2 to 3 days.

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, or when backfill consists of shale mixture. 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 the 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 the 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 3 days 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.

6.5 **Pavement Design**

Where the pavement is to be built on the rooftop of the underground garage/ basement, a sufficient granular base and adequate drainage must be provided to prevent frost damage to the pavement. A waterproof membrane must be placed above the structural slab exposed to weathering to prevent water leakage, as well as to protect the reinforcing steel bars against brine corrosion.

The recommended pavement structure to be placed on top of the underground garage is presented in Table 5.

Table 5 - Pavement Design (Roof of Underground Garage)

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	60	HL-8
Granular Base	250	20-mm Crusher-Run Limestone
Granular Sub-base	100	Free-Draining Sand Fill

The granular bases should be compacted to 100% of their maximum Standard Proctor dry density.

For the on-grade portion of the pavement, such as for local roads and access from local roads, the recommended pavement structure is given in Table 6.

Table 6 - Pavement Design (On-Grade Pavement)

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	65	HL-8
Granular Base	150	20-mm Crusher-Run Limestone
Granular Sub-base Light Duty Heavy Duty	 250 350	50-mm Crusher-Run Limestone

In preparation of the subgrade, the peat and alluvial deposits should be removed, and the subgrade surface should be proof-rolled. The existing earth fill and soft/loose soils must be subexcavated, sorted free of topsoil inclusions or any other deleterious materials, aerated and properly compacted. Any soft or loose subgrade, or earth fill in which the topsoil inclusions and/or other deleterious materials cannot be sorted, should be subexcavated and replaced by properly compacted, organic-free earth fill or granular materials. All the granular bases should be compacted to their maximum Standard Proctor dry density.

Along the perimeter where runoff may drain onto the pavement, a swale or an intercept subdrain system should be installed to prevent infiltrating precipitation from seeping into the granular bases (since this may inflict frost damage on the flexible pavement). Subdrains consisting of filter-wrapped weepers should also be installed, and they should be connected to the catch basins and storm manholes in the paved areas. The subdrains should be backfilled with free-draining granular material.

6.6 Sidewalks, Interlocking Stone Pavement and Landscaping

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

6.7 Soil Parameters

The recommended soil parameters for the project design are given in Table 7.

Table 7 - Soil Parameters

Unit Weight and Bulk Factor	Unit Weight (kN/m^3)	Estimated Bulk Factor	
	Bulk	Loose	Compacted
Earth Fill	21.0	1.20	1.00
Silty Clay	22.0	1.30	0.98
Silty Fine Sand	20.0	1.25	1.00
Shale	24.0	1.40	1.10

Table 7 - Soil Parameters (Cont'd)

Lateral Earth Pressure Coefficients	**Active K_a**	**At Rest K_o**	**Passive K_p**
Existing Earth Fill	0.45	0.55	2.22
Silty Clay	0.40	0.50	2.50
Silty Fine Sand	0.33	0.43	3.00
Weathered Shale	0.25	0.35	4.00
Sound Shale	0.20	0.30	5.00
Coefficients of Friction			
Between Concrete and Sound Natural Soils or Shale Bedrock		0.40	
Between Concrete and Granular Base		0.60	
Maximum Allowable Soil Pressure (SLS) For Thrust Block Design			
Sound Natural Soils		200 kPa	

6.8 Excavation

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

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

In the bedrock, a steeper vertical cut can be allowed, provided the bedding plane of the rock is horizontal. Loose rocks protruding from the excavation must be removed for safety.

For excavation purposes, the types of soils are classified in Table 8.

Table 8 - Classification of Soils for Excavation

Material	Type
Shale Bedrock	1
Stiff to very stiff Clay and weathered Shale Bedrock	2
Earth Fill, soft to firm Clay and Sand above groundwater	3
Sand below groundwater	4

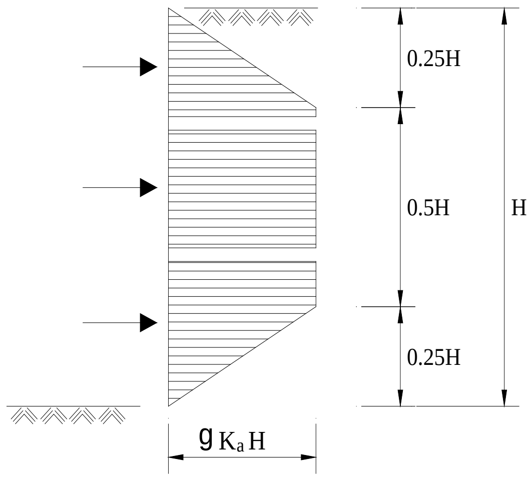
The shale bedrock will require extra effort for excavation using mechanical means, and a rock-ripper will be required to facilitate the excavation. This method can generally be employed to excavate the shale to a depth of 3.0 m below the bedrock surface. Excavation into the sound shale can be carried out by a heavy-duty backhoe equipped with a pneumatic chisel. However, to expedite the cutting process, blasting can be considered, in which case an expert must be consulted to determine the precautionary measures necessary to prevent damage to the existing surrounding buildings and buried structures from the blasting shock waves.

As previously discussed, the groundwater yield from the clay will be small and limited, whereas the yield in the sand will be moderate to appreciable, depending on its extent. In general, groundwater can be controlled by pumping from sumps.

However, in the water-bearing sand, sand fill and rock fill, dewatering by a well-point dewatering system may be necessary. The shale contains occasional pockets of groundwater which are trapped in the rock fissures and may sometimes be under moderate subterranean artesian pressure. Upon release through excavation, this water will likely drain readily with limited yield.

If shoring is required for braced excavation, it should be designed using the lateral earth pressure distribution for the revealed soils illustrated in Diagram 2.

Diagram 2 - Lateral Earth Pressure (Silty Clay and Weathered Shale)



h - Height of Water, if any

H - Height of Excavation

γ - Bulk Unit Weight; Refer to Table 7

K_a - Active Lateral Earth Pressure Coefficient; Refer to Table 7

γ_w - Weight of Water

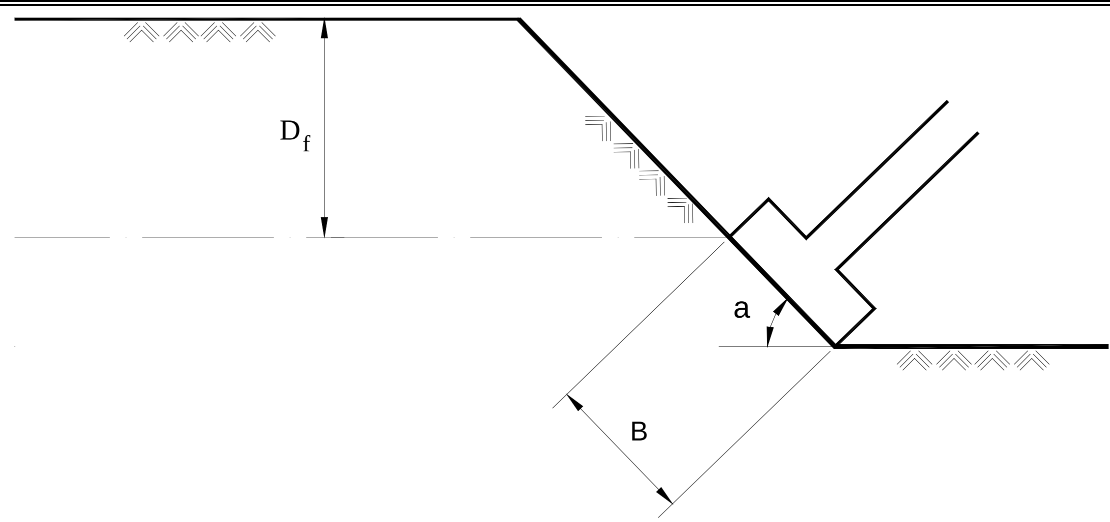
The overburden load of any adjacent existing structures should also be considered in the design of the shoring structures. The load can be converted to overburden pressure, i.e. $H_{total} = H_{excavation} + H_{overburden}$.

If tiebacks are to be used for the shoring structure, the anchors should be embedded into the shale bedrock. Bond resistance for anchors into the sound shale bedrock of 400 kPa can be used for the design of the anchorage embedded in the sound shale bedrock. All the tieback anchors should be proof-loaded to at least 133% of the design load, and at least 1 full scale load test should be carried out on the anchor.

If rakers instead of tiebacks are to be used, they should be designed using the recommended Soil Bearing Pressure given in Table 9.

Table 9 - Soil Pressure for Rakers

Angle of Raker Inclination (α)	Recommended Soil Pressure (kPa)			
	Weathered Shale		Sound Shale	
	Df/B=0	Df/B=1	Df/B=20	Df/B=1
30°	500	800	750	1200
45°	450	750	700	1000
60°	400	650	600	800



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 no tests have been carried out to determine whether environmental contaminants are present in the soils. 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 Shoreline Towers Inc., and for review by its designated consultants and government agencies. The material in it reflects the judgement of Oday Wade'e, EIT, M.Sc., Mumta Mistry, B.Sc., 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.

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LICENSED PROFESSIONAL ENGINEER
CHUN C. B. SUN
Oct 20, 2014
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	1 inch = 25.4 mm
1lb = 0.454 kg	1ksf = 47.88 kPa

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CONSULTING ENGINEERS
GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE

JOB NO: 1203-S013

LOG OF BOREHOLE NO: 1

FIGURE NO: 1

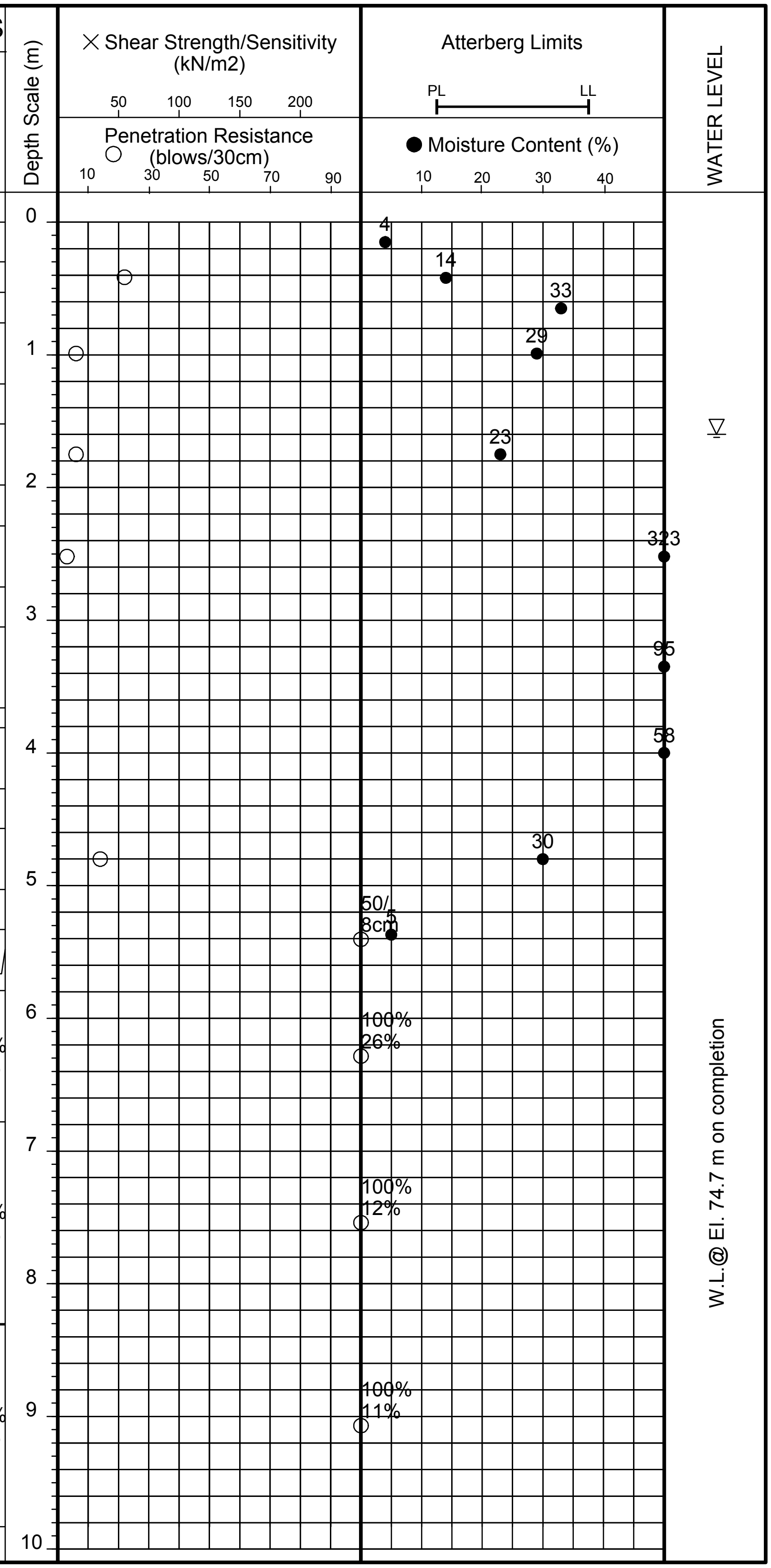
JOB DESCRIPTION: Proposed Condominium with 3-Level Underground Parking

JOB LOCATION: Part of Lots 377, 378 and 379
Behind 2313 and 2323 Lakeshore Boulevard West, City of Toronto

METHOD OF BORING: Flight-Auger

DATE: March 28, 2012

Depth / Elev. (m)	SOIL DESCRIPTION	Samples: Number	Type	N-Value
0.0	Pavement Surface			
76.3	**65 mm ASPHALTIC CONCRETE**	1	AS	-
	180 mm GRANULAR, Fill			
	Brown **SILTY CLAY, Fill** occ. sand seams and layers traces of brick fragments and organic substances	2A	DO	22
		2B	DO	-
		3	DO	6
1.2 / 75.1	Grey, loose **SILTY FINE SAND**	4	DO	6
2.3 / 74.0	Black **PEAT** with alluvium and silty clay deposits	5	DO	3
		6	AS	WH
3.4 / 72.9	Very soft to stiff **SILTY CLAY** with plant debris alluvial deposit a trace to some sand occ. weathered shale fragments (blackish-brown / grey)	7	DO	WH
		8	DO	14
5.0 / 71.3	Grey **SHALE BEDROCK** (weathered)	9	DO	50/ 8
		10	RC RQD	100% 26%
		11	RC RQD	100% 12%
		12	RC RQD	100% 11%
9.8 / 66.5	**END OF BOREHOLE**			



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JOB NO: 1203-S013

LOG OF BOREHOLE NO: 2

FIGURE NO: 2

JOB DESCRIPTION: Proposed Condominium with 3-Level Underground Parking

JOB LOCATION: Part of Lots 377, 378 and 379
Behind 2313 and 2323 Lakeshore Boulevard West, City of Toronto

METHOD OF BORING: Flight-Auger

DATE: March 28, 2012

Depth / Elev. (m)	SOIL DESCRIPTION	Sample Number	Type	N-Value	Moisture Content (%)
0.0	Pavement Surface				
76.7	**50 mm ASPHALTIC CONCRETE** **200 mm GRANULAR, Fill**	1	AS	-	10
	Brown **SILTY CLAY, Fill** traces of sand and foreign matter occ. sand and silt pockets	2	DO	8	17
		3	DO	7	16
		4	DO	3	19
2.3 / 74.4	Black **FINE TO COARSE SAND, Fill** traces of foreign matters and brick fragments occ. silty clay seams and layers, cobbles and boulders	5	DO	6	26
		6	DO	49	15
3.8 / 72.9	**ROCK, Fill** Hard Augering	7	-	50/0	
4.9 / 71.8	Stiff to very stiff **SILTY CLAY** with wood debris alluvial deposit some broken shale (possible shale reversion)	8	DO	8	12
	weathered dark brown brownish-grey	9	DO	30	13
6.6 / 70.2	Grey **SHALE BEDROCK**				
7.1 / 69.6	**REFUSAL TO AUGERING** **END OF BOREHOLE**	10	DO	50/1	13

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

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

50/ 0cm

50/ 1cm

Atterberg Limits: PL, LL

Moisture Content (%): 10, 20, 30, 40

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

WATER LEVEL

W.L.@ El. 74.9 m on completion

Soil Engineers Ltd.

JOB NO: 1203-S013

LOG OF BOREHOLE NO: 3

FIGURE NO: 3

JOB DESCRIPTION: Proposed Condominium with 3-Level Underground Parking

JOB LOCATION: Part of Lots 377, 378 and 379
Behind 2313 and 2323 Lakeshore Boulevard West, City of Toronto

METHOD OF BORING: Flight-Auger

DATE: March 29, 2012

Depth / Elev. (m)	SOIL DESCRIPTION	Sample Number	Type	N-Value	Penetration Resistance (blows/30cm)	Moisture Content (%)	WATER LEVEL
0.0 / 76.6	Pavement Surface **50 mm ASPHALTIC CONCRETE** **560 mm GRANULAR, Fill**	1	DO	-		5	Dry on completion
		2A	DO	9	9	8	
	Brown, stiff to very stiff **SILTY CLAY** occ. sand and silt seams and layers	2B	DO	-		20	
		3	DO	12	12	19	
		4	DO	18	18	18	
		5	DO	16	16	19	
	brown / grey	6	DO	16	16	15	
		7	DO	14	14	19	
		8A	DO	18	18	13	
6.3 / 70.3	Grey, weathered **SHALE BEDROCK** shale reversion	8B	DO	50/15	50/15cm	7	
6.9 / 69.7	**REFUSAL TO AUGERING** **END OF BOREHOLE**						

Shear Strength/Sensitivity (kN/m2): 50, 100, 150, 200
Penetration Resistance (blows/30cm): 10, 30, 50, 70, 90
Atterberg Limits: PL, LL
Moisture Content (%): 10, 20, 30, 40
Depth Scale (m): 0–8

JOB NO: 1203-S013

LOG OF BOREHOLE NO: 4

FIGURE NO: 4

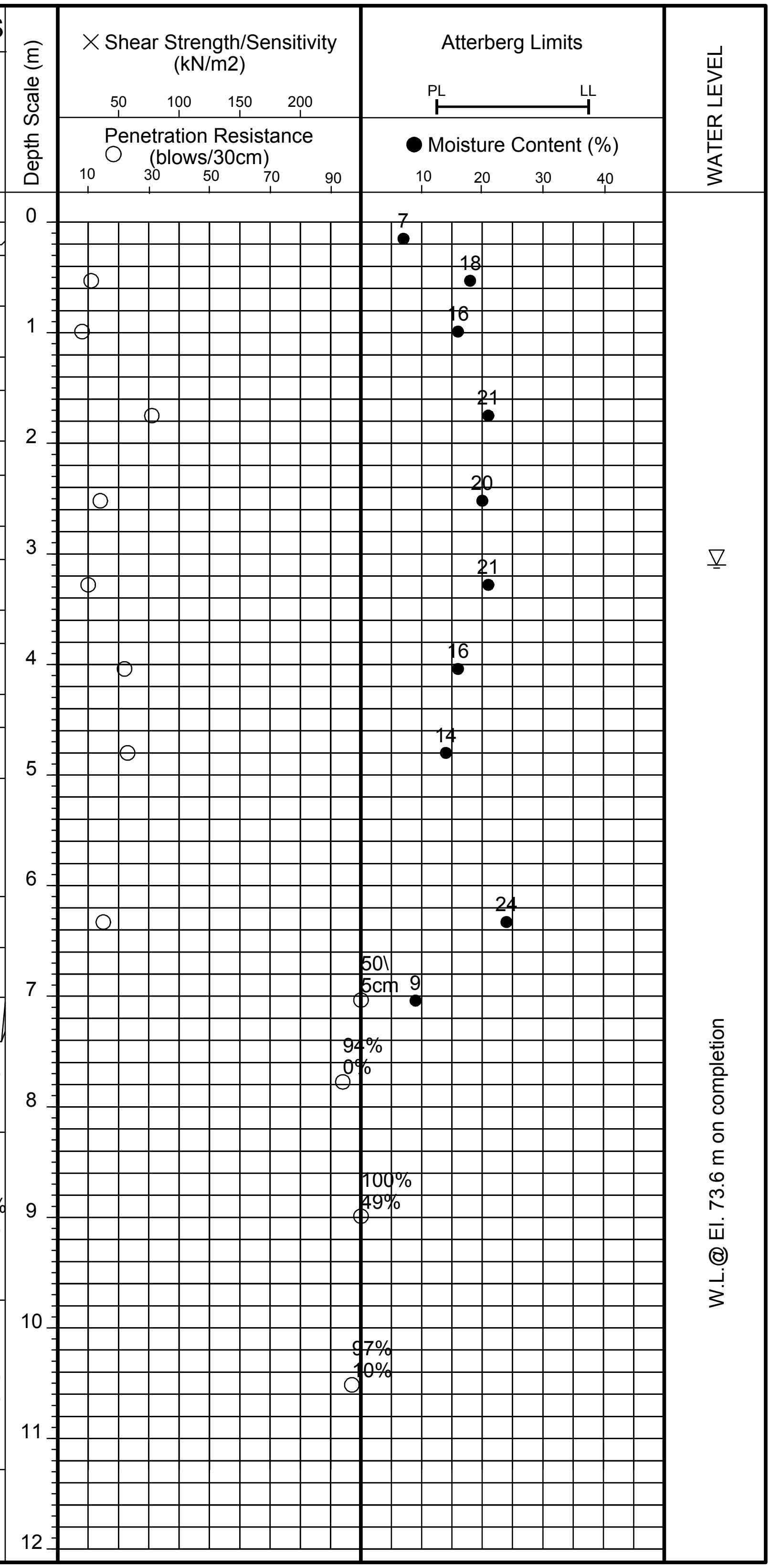
JOB DESCRIPTION: Proposed Condominium with 3-Level Underground Parking

JOB LOCATION: Part of Lots 377, 378 and 379
Behind 2313 and 2323 Lakeshore Boulevard West, City of Toronto

METHOD OF BORING: Flight-Auger

DATE: March 29, 2012

Depth / Elev. (m)	SOIL DESCRIPTION	Number	Type	N-Value
0.0	Pavement Surface			
76.7	**40 mm ASPHALTIC CONCRETE** **125 mm GRANULAR, Fill**	1	AS	-
	Brown and grey **SILTY CLAY, Fill** occ. sand seams and layers traces of brick fragments and organic substances	2	DO	11
		3	DO	8
		4	DO	31
2.3 / 74.4	Grey **FINE TO COARSE SAND, Fill** traces of brick fragments, cobbles and boulders	5	DO	14
		6	DO	10
3.5 / 73.2	Brown, stiff to very stiff **SILTY CLAY** occ. sand and silt seams and layers occ. weathered shale debris	7	DO	22
		8	DO	23
		9	DO	15
6.6 / 70.1	Grey **SHALE BEDROCK**	10	DO	50\ 5
		11	RC RQD	94% 0%
	weathered	12	RC RQD	100% 49%
		13	RC RQD	97% 10%
11.3 / 65.4	**END OF BOREHOLE**			



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JOB NO: 1203-S013

LOG OF BOREHOLE NO: 5

FIGURE NO: 5

JOB DESCRIPTION: Proposed Condominium with 3-Level Underground Parking

JOB LOCATION: Part of Lots 377, 378 and 379
Behind 2313 and 2323 Lakeshore Boulevard West, City of Toronto

METHOD OF BORING: Flight-Auger

DATE: March 30, 2012

Depth / Elev. (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Moisture Content (%)
0.0	Pavement Surface				
76.6	**50 mm ASPHALTIC CONCRETE** **330 mm GRANULAR, Fill**	1	AS	-	5
	Brown **SILTY CLAY, Fill** occ. sand seams and layers a trace of brick fragments	2A	DO	7	21
		2B	DO	-	22
		3	DO	9	20
		4	DO	37	14
2.3 / 74.3	Dark brown **PEAT** occ. alluvium deposits and decay wood	5	DO	3	77
3.1 / 73.5	Grey to brownish-grey, firm to very stiff **SILTY CLAY** occ. sand and silt seams and layers	6	DO	7	20
		7	DO	26	15
		8	DO	18	15
7.6 / 69.0	Grey, weathered **SHALE BEDROCK**	9	DO	65	10
8.5 / 68.1	**REFUSAL TO AUGERING** **END OF BOREHOLE**				

Depth Scale (m): 0 1 2 3 4 5 6 7 8 9 10

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

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

Atterberg Limits: PL LL

● Moisture Content (%): 10 20 30 40

WATER LEVEL: W.L.@ El. 73.2 m on completion

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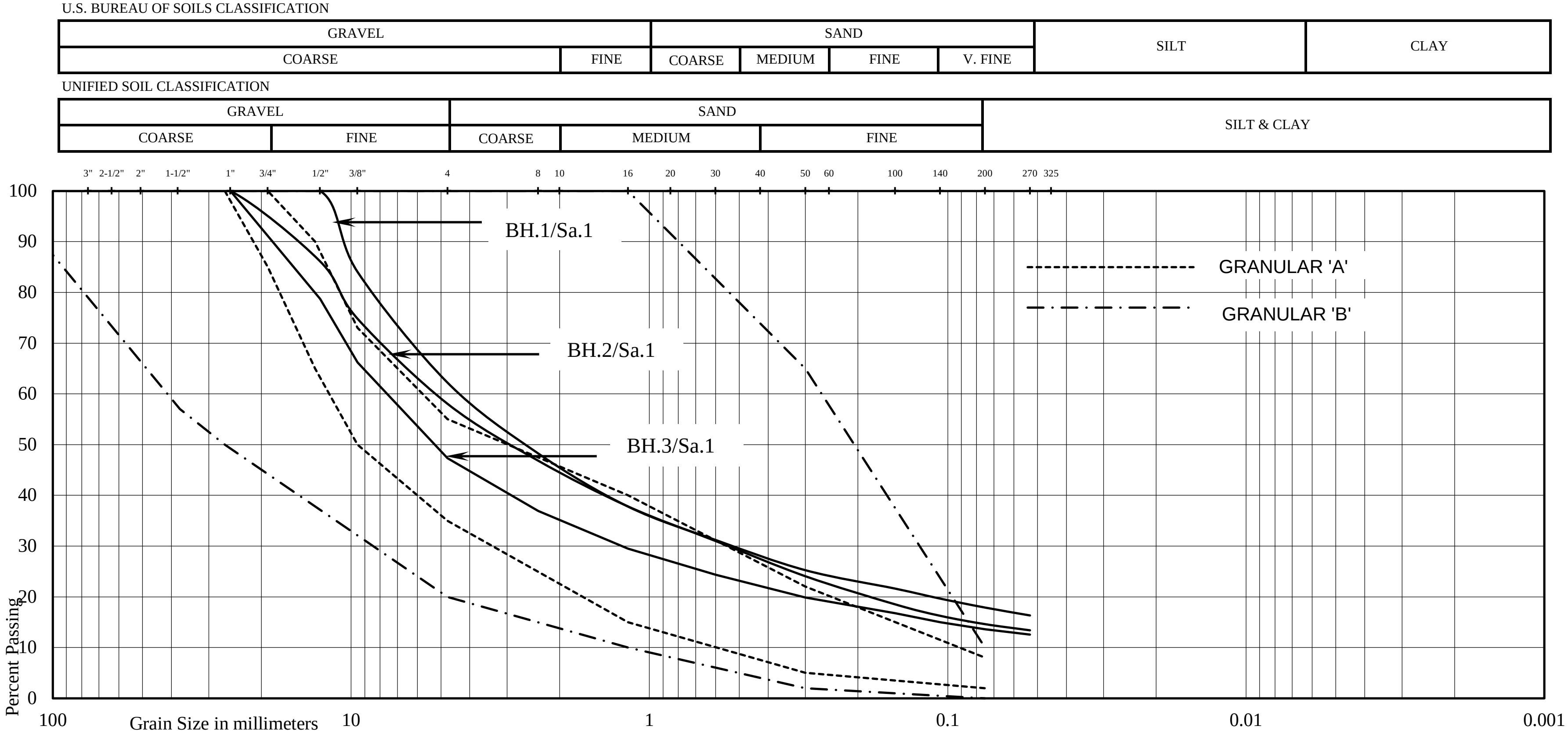
GRAIN SIZE DISTRIBUTION

Reference No: 1203-S013



Project:	Proposed Condominium with 3-Level Underground Parking				BH./Sa.	1/1	2/1	3/1
Location:	Part of Lots 377, 378 and 379				Liquid Limit (%) =	-	-	-
	Behind 2313 and 2323 Lake Shore Boulevard West, City of Toronto				Plastic Limit (%) =	-	-	-
Borehole No:	1	2	3		Plasticity Index (%) =	-	-	-
Sample No:	1	1	1		Moisture Content (%) =	4	10	5
Depth (m):	0.3	0.1	0.1		Estimated Permeability			
Elevation (m):	76.0	76.6	76.5		(cm./sec.) =	10^{-2}	10^{-2}	10^{-2}

Classification of Sample [& Group Symbol]: GRANULAR, Fill

Figure: 6

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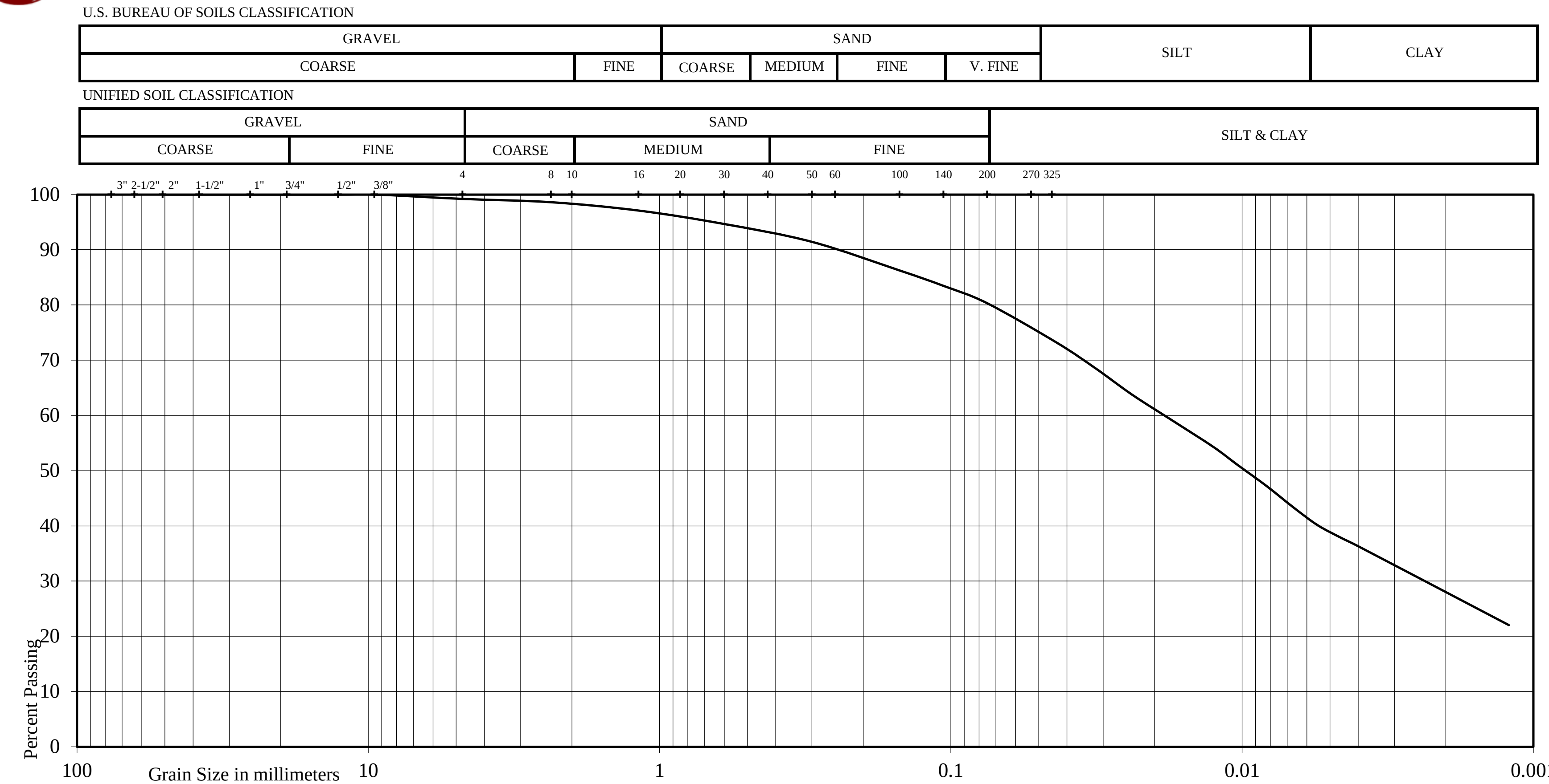
GRAIN SIZE DISTRIBUTION

Reference No: 1203-S013



Project: Proposed Condominium with 3-Level Underground Parking

Location: Part of Lots 377, 378 and 379

Behind 2313 and 2323 Lake Shore Boulevard West, City of Toronto

Borehole No: 2

Sample No: 3

Depth (m): 1.0

Elevation (m): 75.7

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 16

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

Classification of Sample [& Group Symbol]: SILTY CLAY, Fill

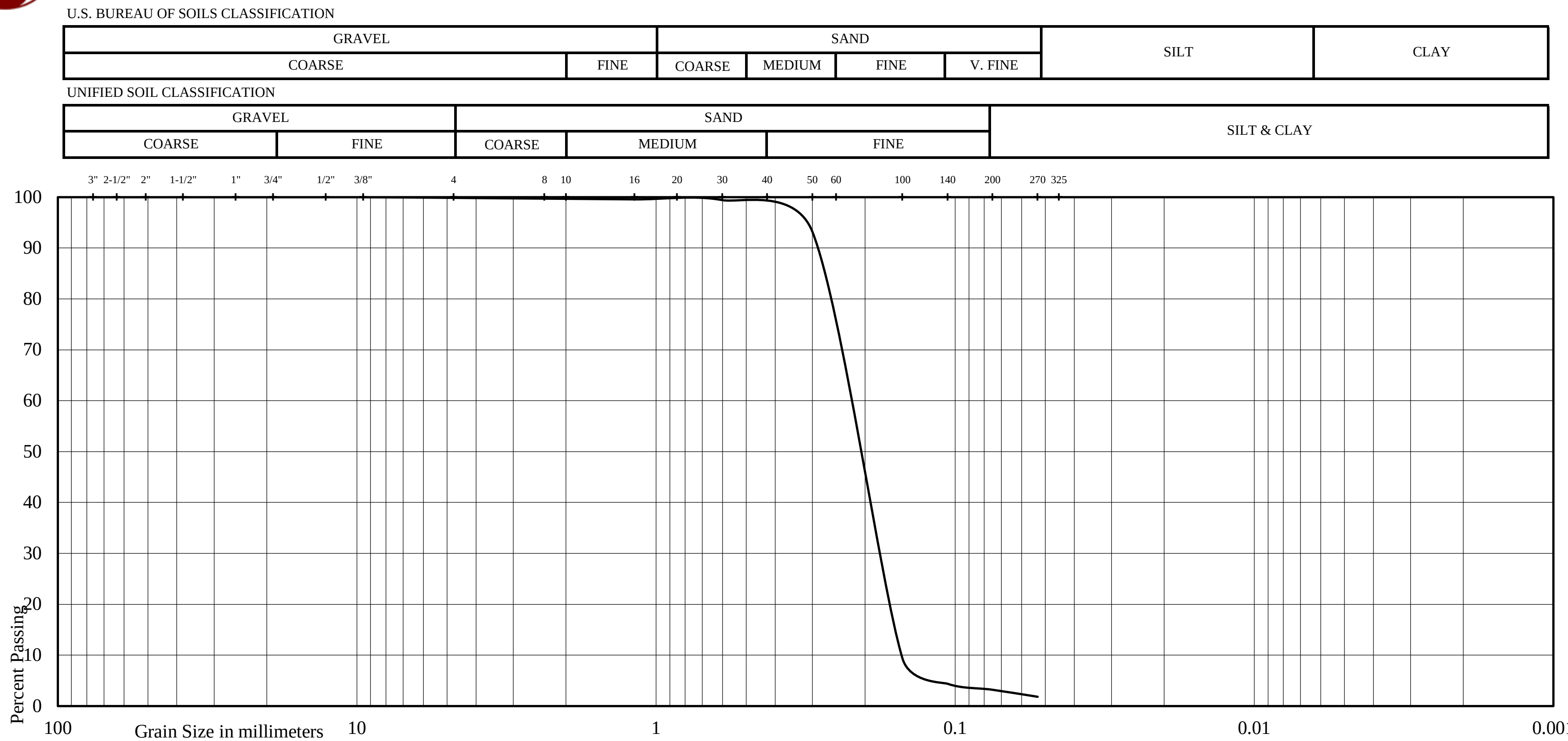
some sand

Figure: 7

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 1203-S013



Project:	Proposed Condominium with 3-Level Underground Parking		
Location:	Part of Lots 377, 378 and 379	Liquid Limit (%) =	-
	Behind 2313 and 2323 Lake Shore Boulevard West, City of Toronto	Plastic Limit (%) =	-
Borehole No:	1	Plasticity Index (%) =	-
Sample No:	4	Moisture Content (%) =	23
Depth (m):	1.7	Estimated Permeability	
Elevation (m):	74.6	(cm./sec.) =	10^{-4}

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

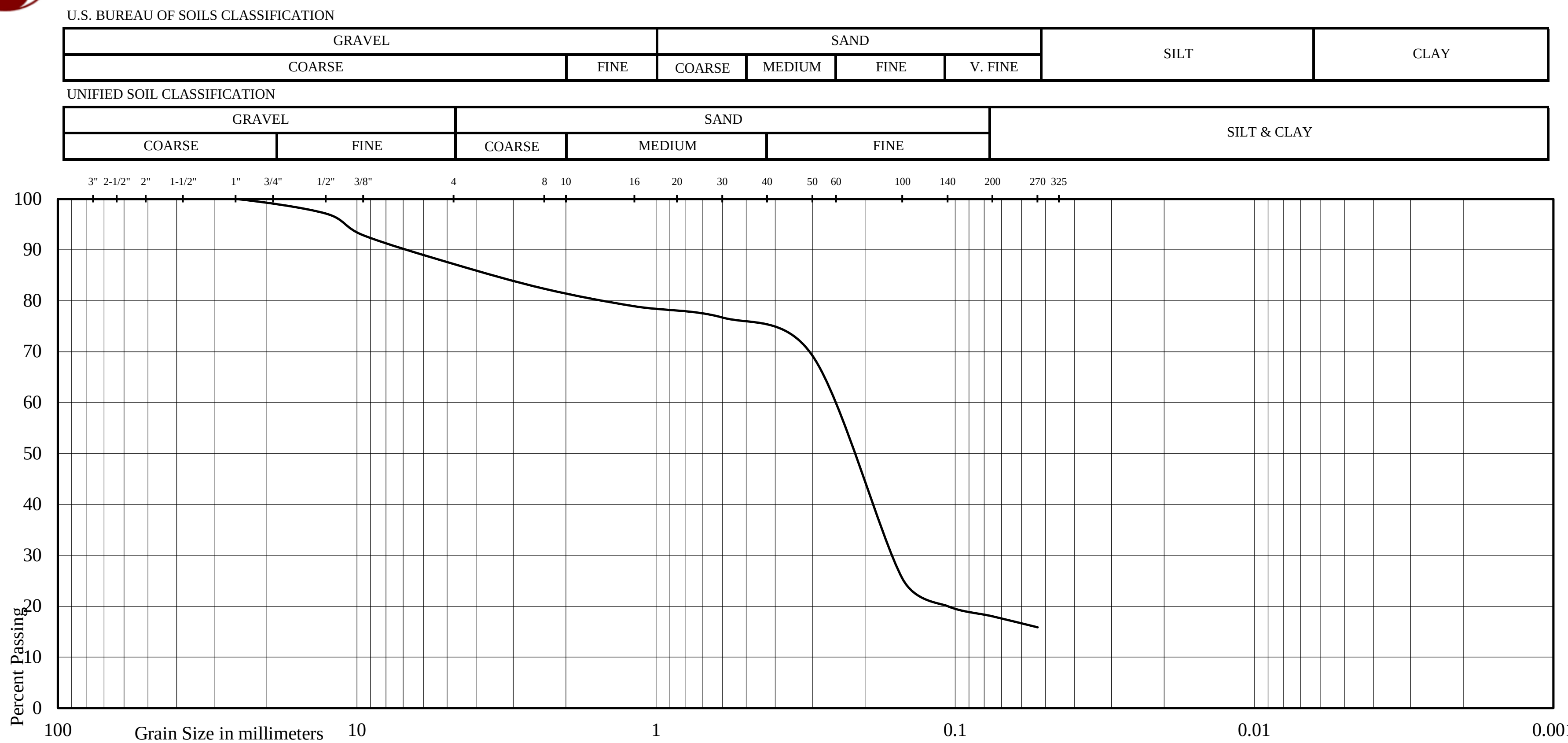
a trace of silt

Figure: 8

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 1203-S013



Project:	Proposed Condominium with 3-Level Underground Parking		
Location:	Part of Lots 377, 378 and 379	Liquid Limit (%) =	-
	Behind 2313 and 2323 Lake Shore Boulevard West, City of Toronto	Plastic Limit (%) =	-
Borehole No:	4	Plasticity Index (%) =	-
Sample No:	5	Moisture Content (%) =	20
Depth (m):	2.5	Estimated Permeability	
Elevation (m):	74.2	(cm./sec.) =	10^{-3}

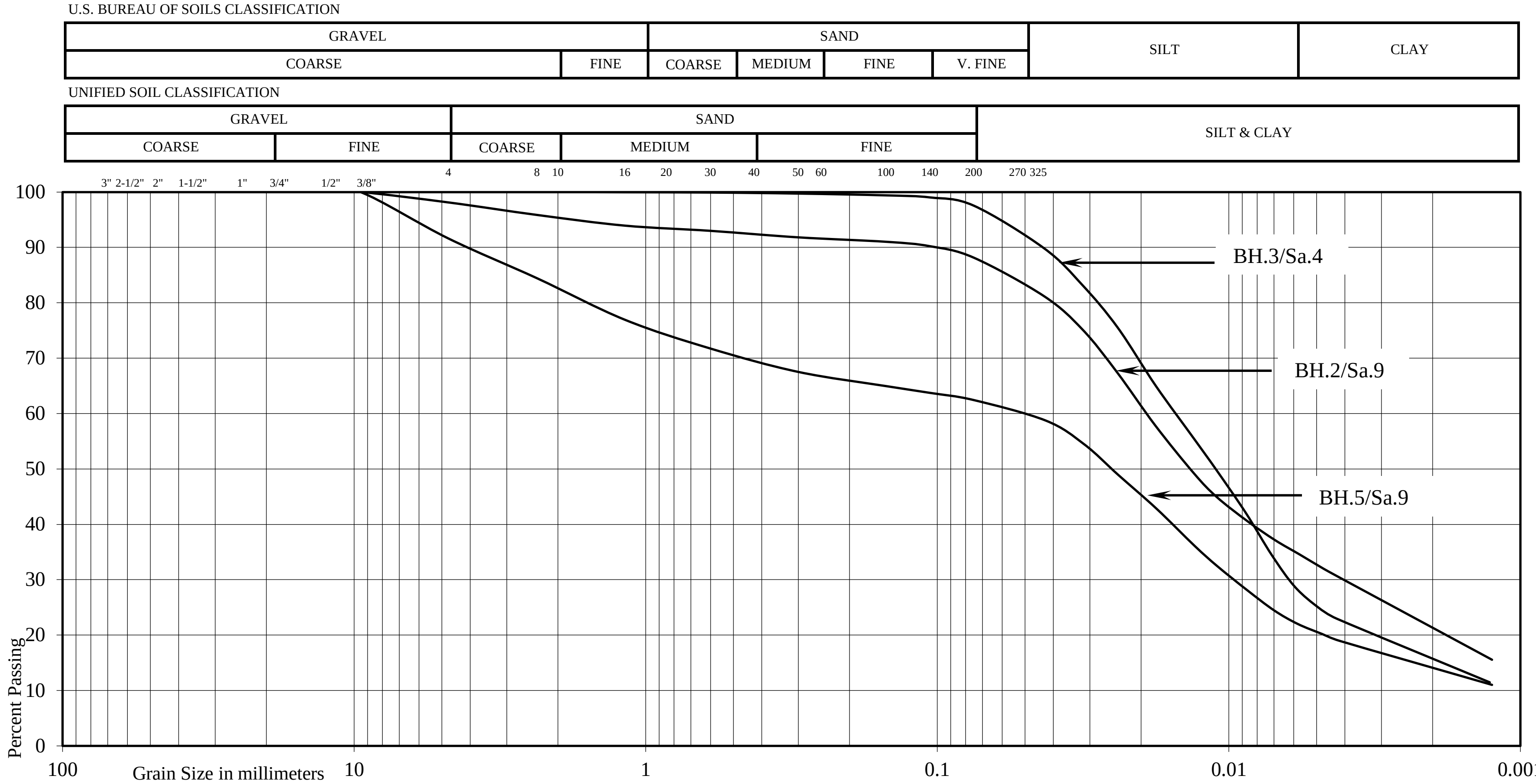
Classification of Sample [& Group Symbol]: FINE TO COARSE SAND
traces of silty clay and gravel

Figure: 9

GRAIN SIZE DISTRIBUTION

Soil Engineers Ltd.

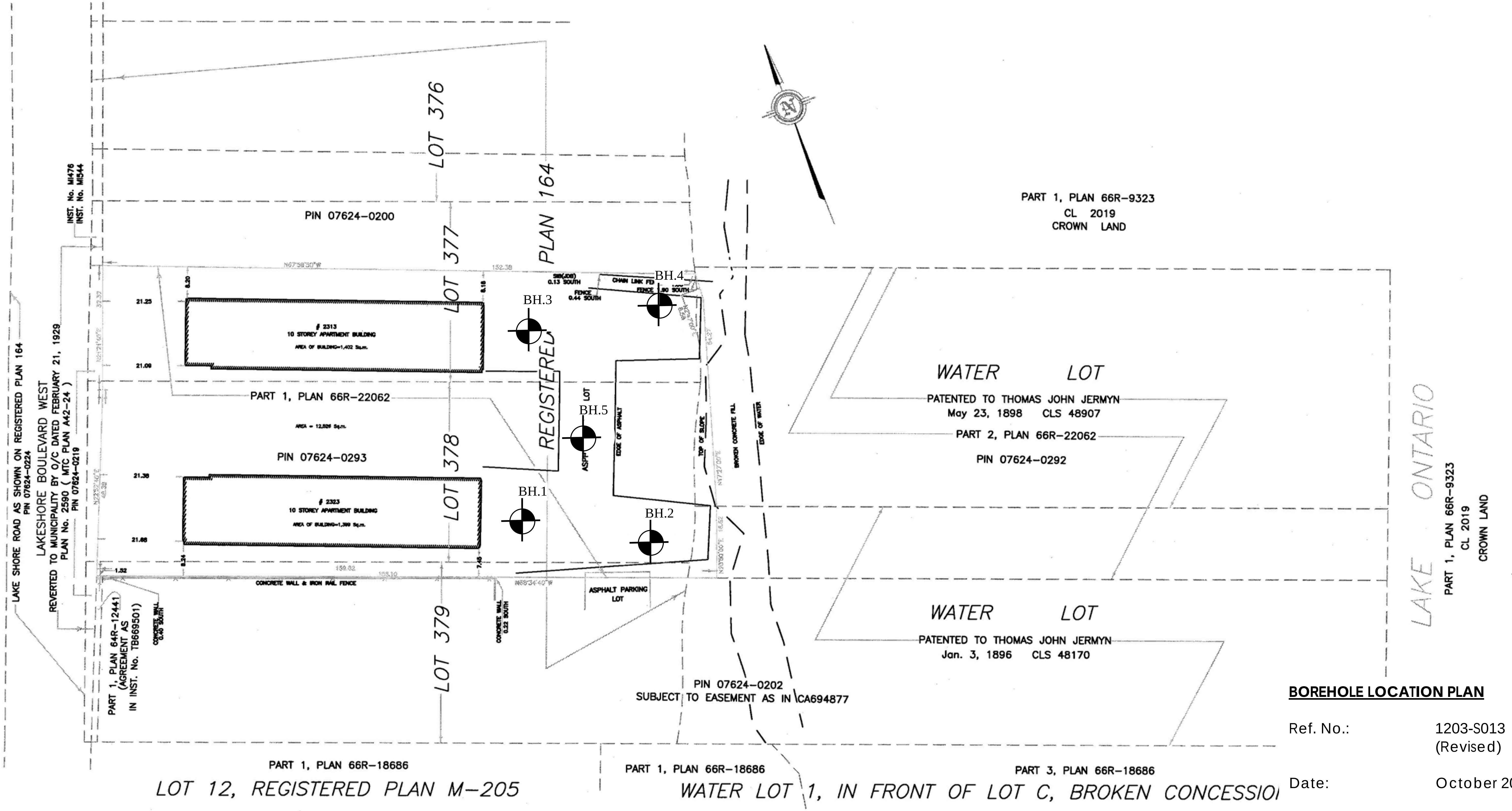
Reference No: 1203-S013



Project:	Proposed Condominium with 3-Level Underground Parking				BH./Sa.	2/9	3/4	5/9
Location:	Part of Lots 377, 378 and 379				Liquid Limit (%) =	31	28	26
	Behind 2313 and 2323 Lake Shore Boulevard West, City of Toronto				Plastic Limit (%) =	17	16	15
Borehole No:	2	3	5		Plasticity Index (%) =	14	12	11
Sample No:	9	4	9		Moisture Content (%) =	13	18	10
Depth (m):	6.3	1.7	7.8		Estimated Permeability			
Elevation (m):	70.4	74.9	68.8		(cm./sec.) =	10^{-7}	10^{-7}	10^{-7}

Classification of Sample [& Group Symbol]:	SILTY CLAY a trace to some sand to sandy, traces of gravel and shale fragments

Figure: 10



BOREHOLE LOCATION PLAN

Ref. No.:	1203-S013 (Revised)
Date:	October 2014
Drawing No.:	1
Scale:	Horiz.: 1:1000

SOIL ENGINEERS LTD.

BH. No.		1		2		3		4		5		
Topsoil (cm)		-		-		-		-		-		
Elevation (m)		76.3		76.7		76.6		76.7		76.6		

SUBSURFACE PROFILE

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Scale: ***Horiz.:*** *N.T.S.*
Vert.: *1:100*

Ref. No.: *1203-S013*
Drawing No. *2*