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A REPORT TO
QUEENSGATE HOMES (BARRIE) INC.

A GEOTECHNICAL INVESTIGATION FOR PROPOSED CONDOMINIUM DEVELOPMENT

681 YONGE STREET
CITY OF BARRIE

REFERENCE NO. 1903-S083

JUNE 2019

DISTRIBUTION

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TABLE OF CONTENTS

TABLES

ENCLOSURES

1.0 INTRODUCTION

In accordance with written authorization dated March 13, 2019, from Mr. Andrew Iacobelli of Queensgate Homes (Barrie) Inc., a geotechnical investigation was carried out at 681 Yonge Street in the City of Barrie.

The purpose of the investigation was to reveal the subsurface conditions and determine the engineering properties of the disclosed soils for the design and construction of the proposed 7-storey condominium development.

The geotechnical findings and resulting recommendations for the proposed development are presented in this Report.

2.0 SITE AND PROJECT DESCRIPTION

The City of Barrie is located within the periphery of Lake Simcoe basin where the glacial till has been partly eroded, in places, by glacial Lake Algonquin and filled with glaciolacustrine sand, silt, clay and reworked till.

The subject property, encompassing an area of 1.4 acres, is located on the north side of Yonge Street and has the municipal address of 681 Yonge Street in the City of Barrie. At the time of the investigation, a 1-storey structure occupied the site and another 1-storey structure had previously been demolished. The surface gradient of the site is generally flat and slightly slopes towards the west.

According to the conceptual site plan prepared by Architecture Unfolded, dated January 8, 2019, the existing structure will be demolished for the construction of a 7-storey condominium with 1-level underground parking. The condominium will be provided with municipal services, driveways and on-grade paved parking meeting urban standards.

3.0 FIELD WORK

The field work, consisting of five (5) sampled boreholes to depths ranging from 7.8 to 11.1 m, was conducted on April 18 and 22, 2019, at the locations shown on the Borehole Location Plan, Drawing No. 1.

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

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

The ground elevation at each borehole location was determined with reference to the site bench mark shown on Drawing No.1 that has the geodetic elevation of 256.9 m.

4.0 SUBSURFACE CONDITIONS

The investigation revealed that below a veneer of topsoil 22 cm thick and/or a layer of earth fill extending to depths of 1.4 to 1.6 m, the site is underlain by strata of silty sand till and sandy silt till embedded with sand layers. Cobbles and boulders are encountered in the tills, in places.

Detailed descriptions of the subsurface profile are shown on the Logs of Boreholes, Figures 1 to 5, inclusive. The revealed stratigraphy is plotted on the Drawing No. 2, and the engineering properties of the disclosed soils are discussed herein.

4.1 **Topsoil** (Boreholes 2 and 3)

The revealed topsoil veneer is approximately 22 cm thick. It is dark brown in colour, indicating an appreciable amount of roots and humus. These materials are unstable and compressible under loads; therefore, the topsoil is void of engineering value and can only be used for general landscaping purposes.

4.2 **Earth Fill** (All Boreholes)

A layer of earth fill was encountered below the topsoil veneer or at the ground surface, extending to a depth of 1.4 to 1.6 m below the prevailing ground surface. It consists of a mixture of silty clay and silty sand with traces of gravel and topsoil inclusions.

The natural water content was determined, and the results are plotted on the Borehole Logs; the values range from 13% to 33%, with a median of 18%, indicating that the fill is very moist to wet condition, which corresponds to our sample examinations. The high moisture may be a result of the inclusion of topsoil and organics.

Due to its unknown history and generally loose density, the earth fill is considered to be unsuitable for supporting structures sensitive to settlement. For structural use, the fill must be subexcavated, inspected, sorted free of topsoil and any deleterious material, if detected, aerated and properly compacted.

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 was completely stripped. This should be further assessed by laboratory testing and/or test pits.

4.3 **Sandy Silt Till/Silty Sand Till** (All Boreholes)

Native silty sand till and sandy silt till deposits were encountered below the earth fill in the boreholes. They consist of a random mixture of particle sizes ranging from clay to gravel, with sand and silt being the dominant fraction. They are amorphous in structure showing the deposits are glacial tills, part of which has been reworked by the glacial lake. Tactile examinations of the soil samples indicated that the tills are slightly cemented, displaying some cohesion.

The obtained ‘N’ values of the till deposits range from 9 to over 100, with a median of 58 blows per 30 cm penetration, indicating the relative density of the tills is loose to very dense, being generally very dense. The loose soil is likely caused by the weathering process that occurs with a depth of 1.6 to 1.8 m from grade.

The natural water content values were determined, and the results are plotted on the Borehole Logs; the values range from 7% to 14%, with a median of 9%, indicating the tills are moist to very moist, being generally moist.

Grain size analyses were performed on 2 representative samples of the sand till deposit. The results are plotted on Figure 6.

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

- Moderately high frost susceptible.
- Moderately water erodible.
- Relatively low to low permeability, with an estimated coefficient of permeability of 10^{-5} to 10^{-6} cm/sec, and runoff coefficients of:

Slope	
0% - 2%	0.11 to 0.15
2% - 6%	0.16 to 0.20
6% +	0.23 to 0.28

- Frictional soils, their shear strength is primarily derived from internal friction, and is augmented by cementation.
- The tills will be stable in steep cuts; however, under prolonged exposure, localized sheet collapse will likely occur.
- Fair pavement-supportive materials, 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.

4.4 **Sand** (Borehole 5)

The sand deposit was encountered beneath the silt till, at a depth of 7.0 m and extends to the maximum investigated depth. It is fine to medium grained with gravel and a trace of silt.

The natural water content of the samples was found to be 7%. Given its pervious nature, water could have been drained from the sample as it was taken. Hence, this value does not represent the in-situ water content.

The obtained ‘N’ value is over 100 blows per 30 cm of penetration, showing the deposit is very dense in relative density.

The following engineering properties are deduced:

- Low frost susceptibility and moderately 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

- Frictional soil, the shear strength is derived from internal friction and is soil density dependent.
- In excavation, the deposits will slough, run under seepage pressure and boil with a piezometric head of about 0.3 m.
- Good pavement-supportive material, with an estimated CBR value of 25%.

- Low corrosivity to buried metal, with an estimated electrical resistivity of 6500 ohm·cm.

5.0 GROUNDWATER CONDITIONS

The boreholes were checked for the presence of groundwater or the occurrence of cave-in upon completion of the field work. The results are presented on the borehole logs and are summarized in Table 1.

Table 1 - Groundwater Levels

Borehole No.	Borehole Elevation (m)	Borehole Depth	Measure Groundwater Level/ Cave-in Level*	
			Depth (m)	Elevation (m)
1	256.0	11.1	6.7	249.3
2	257.4	11.1	3.0/7.6*	254.4/249.8*
3	257.3	10.9	8.2*	249.1*
4	256.1	9.5	1.8/6.4*	254.3/249.7*
5	256.9	7.8	6.1*	250.8*

Groundwater and cave-in were encountered within the tills at depths ranging from 1.8 to 8.2 m below the prevailing ground surface. It is our opinion that the recorded groundwater represents the groundwater regime of the site. In excavation, seepage from groundwater and infiltration of surface water will be expected. It can be drained and removed by pumping from sumps.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation revealed that below a veneer of topsoil, 22 cm thick, and/or a layer of earth fill extending to depths of 1.4 to 1.6 m, the site is underlain by strata of silty sand till and sandy silt till embedded with occasional sand layers. Cobbles and boulders are encountered in the tills, in places.

Groundwater and cave-in were encountered within the tills at depths ranging from 1.8 to 8.2 m below the prevailing ground surface. It is our opinion that the recorded groundwater represents the groundwater regime of the site. In excavation, seepage from groundwater and infiltration of surface water will be expected. It can be drained and removed by pumping from sumps.

The proposed building will be a 7-storey condominium with 1-level underground parking. The geotechnical findings warranting special consideration for the proposed development are presented below:

1. The topsoil should be removed for the project development. It is compressible and contains an appreciable amount of organics and humus. Therefore, the topsoil is void of engineering value and can only be used for general landscaping purposes. It must not be buried below any structures or below a depth of 1.2 m from finished grade.
2. The earth fill and weathered soil are not suitable to support any structure sensitive to movement. They must be subexcavated sorted-free of organic or other deleterious materials and properly compacted in layers prior to reuse as structural backfill and/or for the construction of engineered fill on site. If it is impractical to sort the topsoil and other deleterious materials from the fill, the fill must be wasted and replaced with properly compacted inorganic earth fill.

3. After demolition of the existing structure, the cavity should be backfilled with engineered fill.
4. As mentioned, an old structure had previously been demolished and it is unknown whether if the subsurface was completed removed. This may result in complications for the 1-level underground excavation.
5. The sound native soils are suitable for conventional footing construction. The footing must be designed in accordance with the recommended bearing pressures in Section 6.1 and the footing subgrade should be inspected to ensure that the revealed conditions are compatible with the foundation requirements.
6. The construction of a conventional underground parking structure with subsurface drainage collecting the groundwater and dissipating it into the sewage system has to be pre-approved by the Municipality. If the connection for discharge of subsurface water is not approved, the underground parking structure should be designed with a storage system to collect and store the subsurface water.
7. A Class ‘B’ bedding, consisting of compacted 20-mm Crusher-Run Limestone, or equivalent, is recommended for the construction of the underground services.
8. Temporary shoring will be required for the excavation where it is not feasible for open-cut excavation. A pre-construction survey is strongly recommended for the adjacent properties and structures prior to any excavation activities at the site.

The recommendations appropriate for the project described in Section 2.0 are based on the geotechnical findings of this investigation. 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 Foundation

The proposed building will be a 7-storey mixed-use building with 1-level underground parking. It is understood that the founding depth of the building foundation will be approximately 5 m below grade. Based on the borehole findings, the founding subsoil consists of dense to very dense sand till or silt till with sand seams, cobbles and boulders.

The sound natural soils at the founding level are suitable for spread and strip footing construction. The recommended soil bearing capacity is summarized in Table 2 with estimated total and differential settlement at 25 mm and 20 mm, respectively.

Table 2 - Founding Levels

Borehole No.	**Recommended Maximum Allowable Soil Pressure (SLS)/ Factored Ultimate Soil Bearing Pressure (ULS)**			
	250 kPa (SLS)/375 kPa (ULS)		**400 kPa (SLS)/600 kPa (ULS)**	
	Depth (m)	**El. (m)**	**Depth (m)**	**El. (m)**
1	-	-	2.5 or +	253.5 or -
2	3.0 or +	254.4 or -	4.5 or +	252.9 or -
3	2.5 or +	254.8 or -	4.5 or +	252.8 or -
4	2.5 or +	253.6 or -	3.3 or +	252.8 or -
5	1.7 or +	255.2 or -	2.5 or +	254.4 or -

Foundations exposed to weathering or in unheated areas, such as the exterior footings or footings near ventilation shaft and ramp-down driveway, should have at least 1.5 m of earth cover for protection against frost action. For unheated underground parking structure, if the entrance to the garage is kept closed most of the time, the earth cover for footings and pile caps away from entrances and ventilation shaft can be reduced to 0.9 m for perimeter walls and 1.2 m for interior walls and columns.

The footing subgrade should be inspected by a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to ensure that the revealed conditions are compatible with the foundation design requirements.

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).

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

6.2 **Underground Structure**

The perimeter walls of the underground structure should be designed to sustain a lateral earth pressure calculated using the soil parameters stated in Section 6.8. Any applicable surcharge loads adjacent to the proposed building and uplift forces must also be considered in the design of the underground structure.

In conventional design, the perimeter walls should be dampproofed and provided with a perimeter subdrain at the wall base (Drawing No. 3). Backfill of open excavation should be free-draining granular material unless prefabricated drainage board is installed over the entire wall below grade. At the shoring location, prefabricated drainage board, such as Miradrain 6000 or equivalent, must be provided on the perimeter walls, between the shoring wall and the cast-in-place foundation wall (Drawing No. 4).

The perimeter drains should be installed with a positive gradient, connecting into frost-free sump-pits, in separate drain pipes, where water can be discharged into the municipal storm sewer system to maintain the water level below the slab-on-grade. All the subdrains should be encased in a fabric filter to protect them against blockage by silting.

If the Municipality does not allow any discharge of subsurface water into the municipal system, a separate storage system should be provided.

The elevator pit, which normally extends a few metres below the floor level, should be designed as a submerged ‘tank’ structure with waterproofed pit walls and pit floor.

6.3 Slab-On-Grade Construction

The subgrade for conventional slab-on-grade construction should consist of sound natural soils or properly compacted inorganic earth fill. In preparation of the subgrade, any loose soil and deleterious materials detected must be removed.

The slab floor should be constructed on a granular base, consisting of 20-mm Crusher-Run Limestone, or equivalent, 20 cm thick, compacted to its maximum Standard Proctor dry density. A Modulus of Subgrade Reaction of 30 MPa/m can be used for the design of the slab-on-grade.

At the exterior, the slab-on-grade or sidewalk should be designed to tolerate frost heave. The grading around the slab-on-grade must be such that it directs runoff away from the surface to minimize the frost heave phenomenon generally associated with the disclosed soils.

At the garage entrances, the subgrade should be properly insulated, or the subgrade material should be replaced with 1.5 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 entrance and in areas of close proximity to air shafts should be insulated, and the insulation should extend 1.5 m internally. This measure is to prevent frost action induced by cold drafts. The exterior grade should slope away from the building. This is to prevent runoff from ponding in the areas adjacent to the underground garage.

6.4 **Underground Services**

The subgrade for the underground services should consists of sound natural soils or properly compacted organic-free earth fill. Where topsoil, organic earth fill or badly weathered soils are encountered, it should be subexcavated and replaced with bedding material compacted to at least 95% or + of its Standard Proctor compaction.

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

In order to prevent pipe floatation when the underground services 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.

The pipe joints should be leak-proof, or wrapped with a waterproof membrane to prevent subgrade migration through leakage at joints resulted from inadvertent faulty installation. Openings to subdrains and catch basins should be shielded with a fabric filter to prevent blockage by silting.

The insitu soils have moderately low corrosivity to buried metal. In determining the mode of protection, an electrical resistivity of 5000 ohm·cm should be used. This, however, should be confirmed by testing the soil along the water main alignment at the time of services construction.

6.5 **Sidewalks and Interlocking Stone Pavement**

Interlocking stone pavement and the sidewalks in areas which are sensitive to frost-induced ground movement, such as in front of building entrances, must be constructed on a free-draining, non-frost-susceptible granular material such as Granular ‘B’. It must extend to 1.5 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 insulated with 15-mm Styrofoam, or equivalent.

6.6 **Backfilling in Trenches and Excavated Areas**

The on-site inorganic soils are generally suitable for use as trench backfill. However, the soils should be sorted free of any topsoil inclusions and other deleterious materials prior to the backfilling.

The backfill should be compacted to at least 95% and 98% of its maximum Standard Proctor dry density below the pavement structure and slab-on-grade, respectively. The material within 1.0 m below the pavement subgrade should be compacted to at least 98% of its maximum Standard Proctor dry density, with the water content 2% to 3% drier than the optimum. This is to provide the required stiffness for pavement construction.

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

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

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

- When construction is carried out in freezing winter weather, allowance should be made for these following conditions. Despite stringent backfill monitoring, frozen soil layers may inadvertently be mixed with the structural trench backfill. Should the in situ soil have a water content on the dry side of the optimum, it would be impossible to wet the soil due to the freezing condition, rendering difficulties in obtaining uniform and proper compaction.

- 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.

6.7 **Pavement Design**

Where the pavement is to be built on a structural slab, such as the underground garage rooftop, 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 the roof of the underground garage is presented in Table 3.

Table 3 - Pavement Design (Roof of Underground Garage)

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	50	HL-8
Granular Base	200	20-mm Crusher-Run Limestone
Granular Sub-base	100	Free-draining Sand Fill

For the on-grade access driveway between the road and the building, the recommended pavement design is presented in Table 4.

Table 4 - Pavement Design (on Grade)

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

In preparation of pavement subgrade, topsoil and compressible material should be removed, and the subgrade surface must be proof-rolled using a heavy roller or loaded dump truck. Any soft spot as identified must be rectified by subexcavation and replacing with dry organic material, compacted to the specified density.

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.

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). The subdrains should consist of filter-wrapped weepers 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.

6.8 **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/m3)**		**Estimated Bulk Factor**	
	Bulk	**Submerged**	**Loose**	**Compacted**
Sandy Silt Till/Silty Sand Till	22.5	12.5	1.30	1.05
Sand	21.5	11.5	1.25	0.98
Lateral Earth Pressure Coefficients		**Active K_a**	**At Rest K_o**	**Passive K_p**
Compacted Earth Fill		0.40	0.55	2.50
Native Tills/Sand		0.30	0.45	3.00
Coefficients of Friction				
Between Concrete and Granular Base			0.5	
Between Concrete and Sound Natural Soils			0.4	
Maximum Allowable Soil Pressure (SLS) For Thrust Block Design				
Engineered Fill and sound natural Soils			75 kPa	

6.9 **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
Native sound Tills	2
Weathered Soils and Earth Fill	3
Soils below groundwater table	4

Where safe opencut excavation is not feasible, a braced shoring will be required. The design parameters for the shoring and our recommendations are provided in the Appendix of this report. The overburden load and the surcharge from any adjacent structures should also be considered in the design of the shoring. Assessment of the adjacent building foundations should be carried out prior to shoring design.

Excavation into the till containing boulders may require extra effort and the use of a heavy-duty backhoe. Boulders larger than 15 cm in size are not suitable for structural backfill and/or construction of engineered fill.

In excavation, seepage from groundwater and infiltration of surface water will be expected. It can be controlled by pumping from sumps. It can be drained and removed by pumping from sumps.

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

6.10 **Monitoring of Performance**

It is recommended that close monitoring of vertical and lateral movement of the shoring wall should be carried out and frequent site inspections be conducted to ensure that the excavation does not adversely affect the structural stability of the adjacent buildings and the existing underground utilities. Extra bracing or support may be required if any movement is found excessive. The contractor should maintain the shoring to ensure any movement is within the design limit.

Vibration control and pre-construction survey is strongly recommended for the adjacent properties and structures prior to any excavation activities at the site. Soil Engineers Ltd. can provide further advice or undertake the vibration control and pre-construction survey as necessary.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of Queensgate Homes (Barrie) Inc., for review by the designated consultants, financial institutions, and government agencies. Use of this report is subject to the conditions and limitations of the contractual agreement. The material in the report reflects the judgement of Weida (Daric) Yang, B.A.Sc., and Bernard Lee, P.Eng., in light of the information available to it at the time of preparation. Any use which a Third Party makes of this report, 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.

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LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

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

SAMPLE TYPES

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

PENETRATION RESISTANCE

Dynamic Cone Penetration Resistance:

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

Plotted as '—●—'

Standard Penetration Resistance or 'N' Value:

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

Plotted as '○'

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

SOIL DESCRIPTION

Cohesionless Soils:

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

Cohesive Soils:

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

Method of Determination of Undrained Shear Strength of Cohesive Soils:

x 0.0	Field vane test in borehole; the number denotes the sensitivity to remoulding
△	Laboratory vane test
☐	Compression test in laboratory

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

METRIC CONVERSION FACTORS

1 ft = 0.3048 metres
1lb = 0.454 kg
1 inch = 25.4 mm
1ksf = 47.88 kPa

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JOB NO.: 1903-S083

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

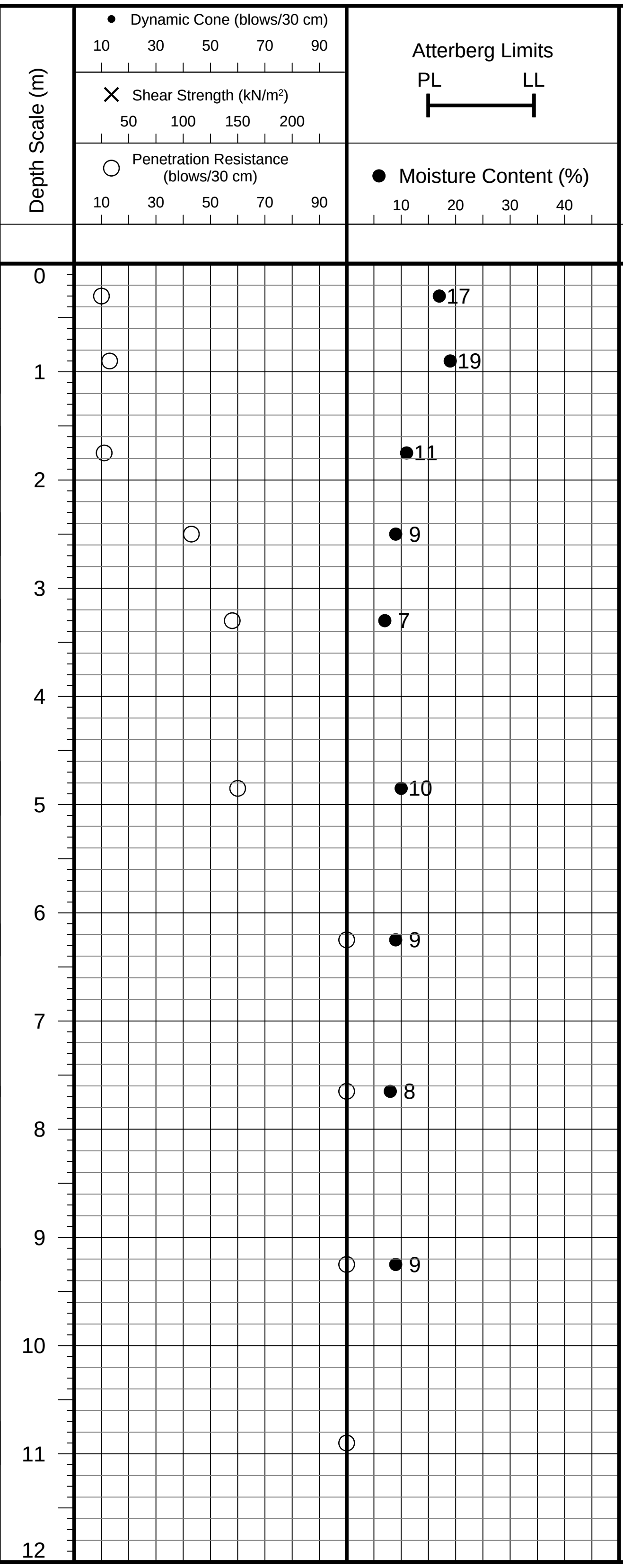
PROJECT DESCRIPTION: Proposed Condominium Development

METHOD OF BORING: Hollow Stem Auger

PROJECT LOCATION: 681 Yonge Street, City of Barrie

DRILLING DATE: April 18, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
256.0	Ground Surface			
0.0	**EARTH FILL** brown, silty sand a trace of gravel occ. topsoil inclusions and organics	1	DO	10
		2	DO	13
254.6 1.4	Compact to very dense **SANDY SILT TILL** trace to some gravel a trace of clay occ. sand seams, cobbles and boulders weathered	3	DO	11
		4	DO	43
		5	DO	58
		6	DO	60
		7	DO	50/15
	brown grey			
		8	DO	50/10
247.5 8.5	Grey, very dense **SILTY SAND TILL** a trace of gravel occ. cobbles and boulders	9	DO	50/10
		10	DO	N/A
244.9 11.1	**END OF BOREHOLE**			



WATER LEVEL

∇ W.L. @ El. of 249.3 m upon completion of drilling

Soil Engineers Ltd.

JOB NO.: 1903-S083

LOG OF BOREHOLE NO.: 2

FIGURE NO.: 2

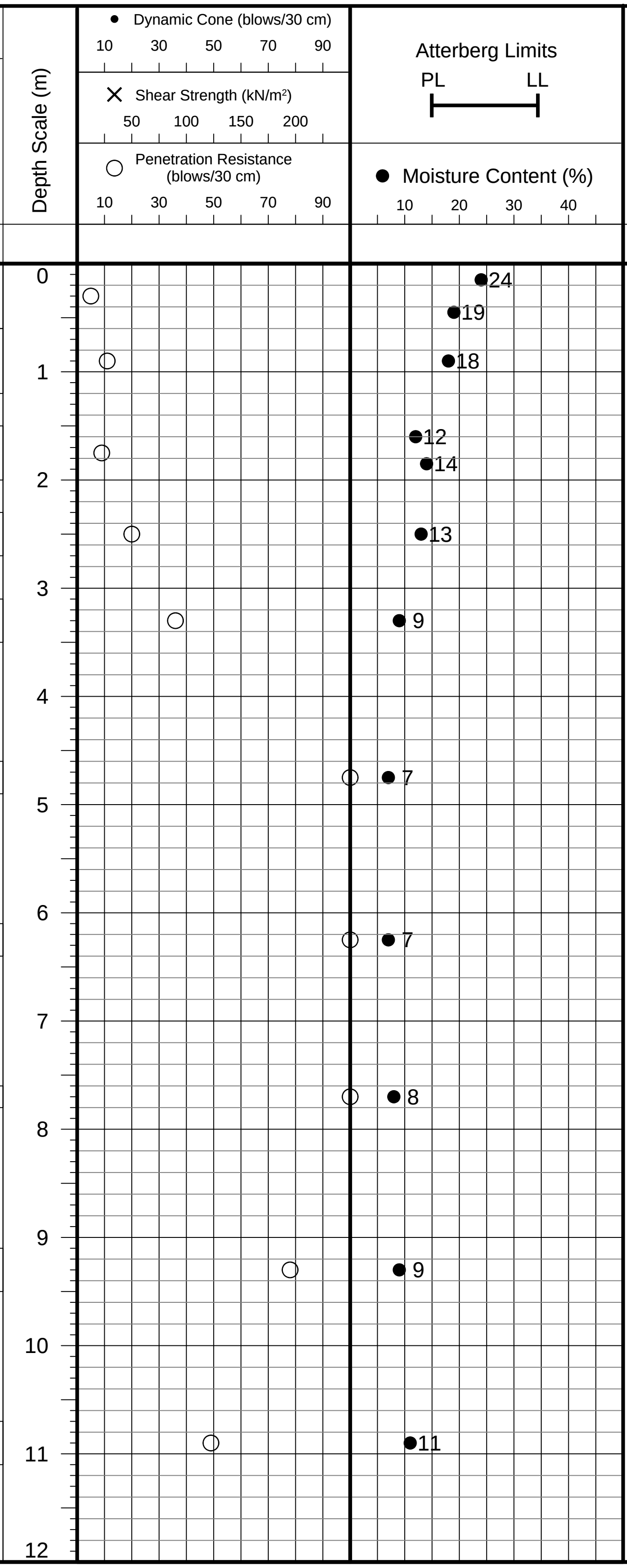
PROJECT DESCRIPTION: Proposed Condominium Development

METHOD OF BORING: Hollow Stem Auger

PROJECT LOCATION: 681 Yonge Street, City of Barrie

DRILLING DATE: April 22, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
257.4	Ground Surface			
0.0	**22 cm Topsoil**	1A	DO	5
0.2	**EARTH FILL** brown, silty clay a trace of sand occ. gravel and topsoil inclusions	1B		
		2	DO	11
256.0 1.4	Loose to very dense **SANDY SILT TILL** traces of clay and gravel occ. sand seams, cobbles and boulders — — weathered	3	DO	9
		4	DO	20
		5	DO	36
		6	DO	50/15
		7	DO	50/15
		8	DO	50/10
		9	DO	78
	— — brown grey			
		10	DO	49
246.3 11.1	**END OF BOREHOLE**			



WATER LEVEL

▽ W.L. @ El. of 254.4 m upon completion of drilling

Cave-in @ El. of 249.8 m upon completion of drilling

Soil Engineers Ltd.

JOB NO.: 1903-S083

LOG OF BOREHOLE NO.: 3

FIGURE NO.: 3

PROJECT DESCRIPTION: Proposed Condominium Development

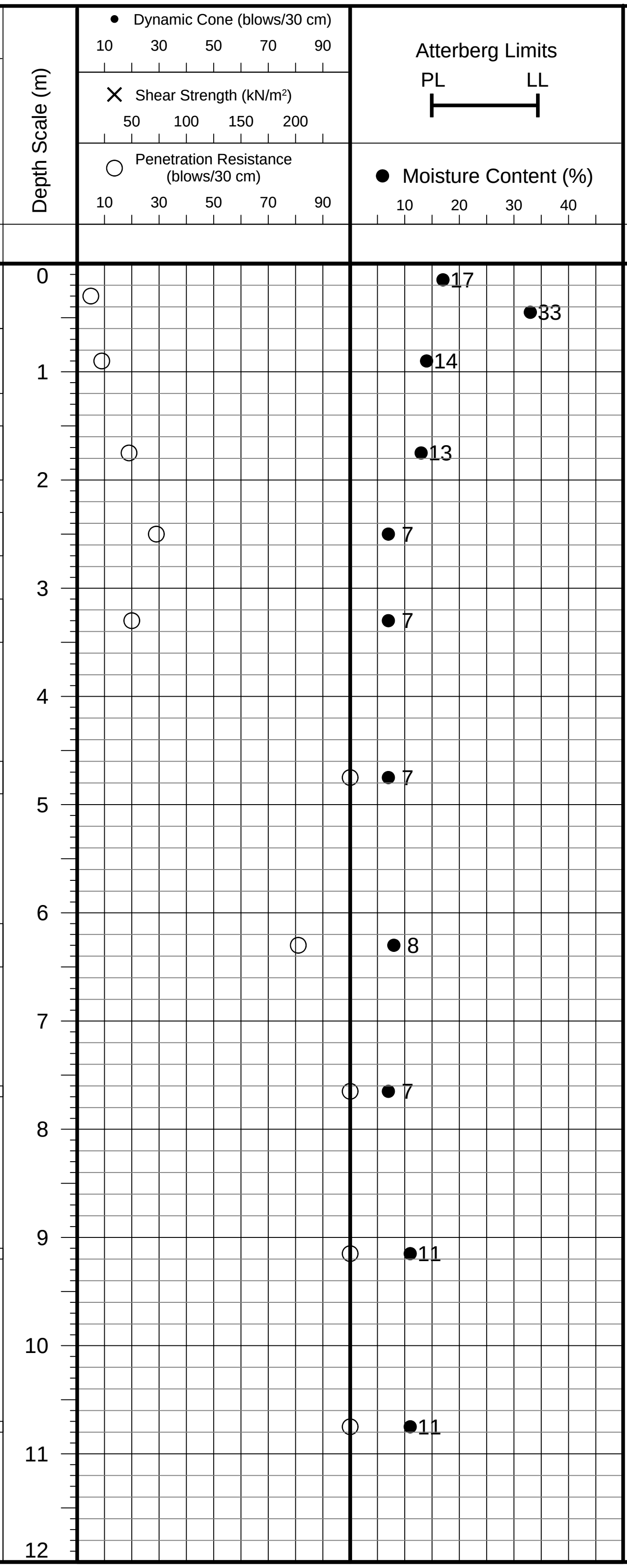
METHOD OF BORING: Hollow Stem Auger

PROJECT LOCATION: 681 Yonge Street, City of Barrie

DRILLING DATE: April 22, 2019

El. (m) Depth (m)	SOIL DESCRIPTION
257.3	Ground Surface
0.0	**22 cm Topsoil**
0.2	**EARTH FILL** brown, silty clay occ. gravel, sand and topsoil inclusions
255.7 1.6	Brown, compact **SANDY SILT TILL** traces of clay and gravel occ. cobbles and boulders
254.4 2.9	Brown, compact to very dense **SILTY SAND TILL** trace to some gravel occ. cobbles and boulders
248.8 8.5	Brown, very dense **SANDY SILT TILL** traces of clay and gravel occ. cobbles and boulders
246.4 10.9	**END OF BOREHOLE**

SAMPLES		
Number	Type	N-Value
1	DO	5
2	DO	9
3	DO	19
4	DO	29
5	DO	20
6	DO	50/15
7	DO	81
8	DO	50/15
9	DO	50/15
10	DO	50/15



WATER LEVEL

Cave-in @ El. of 249.1 m upon completion of drilling

Soil Engineers Ltd.

JOB NO.: 1903-S083

LOG OF BOREHOLE NO.: 4

FIGURE NO.: 4

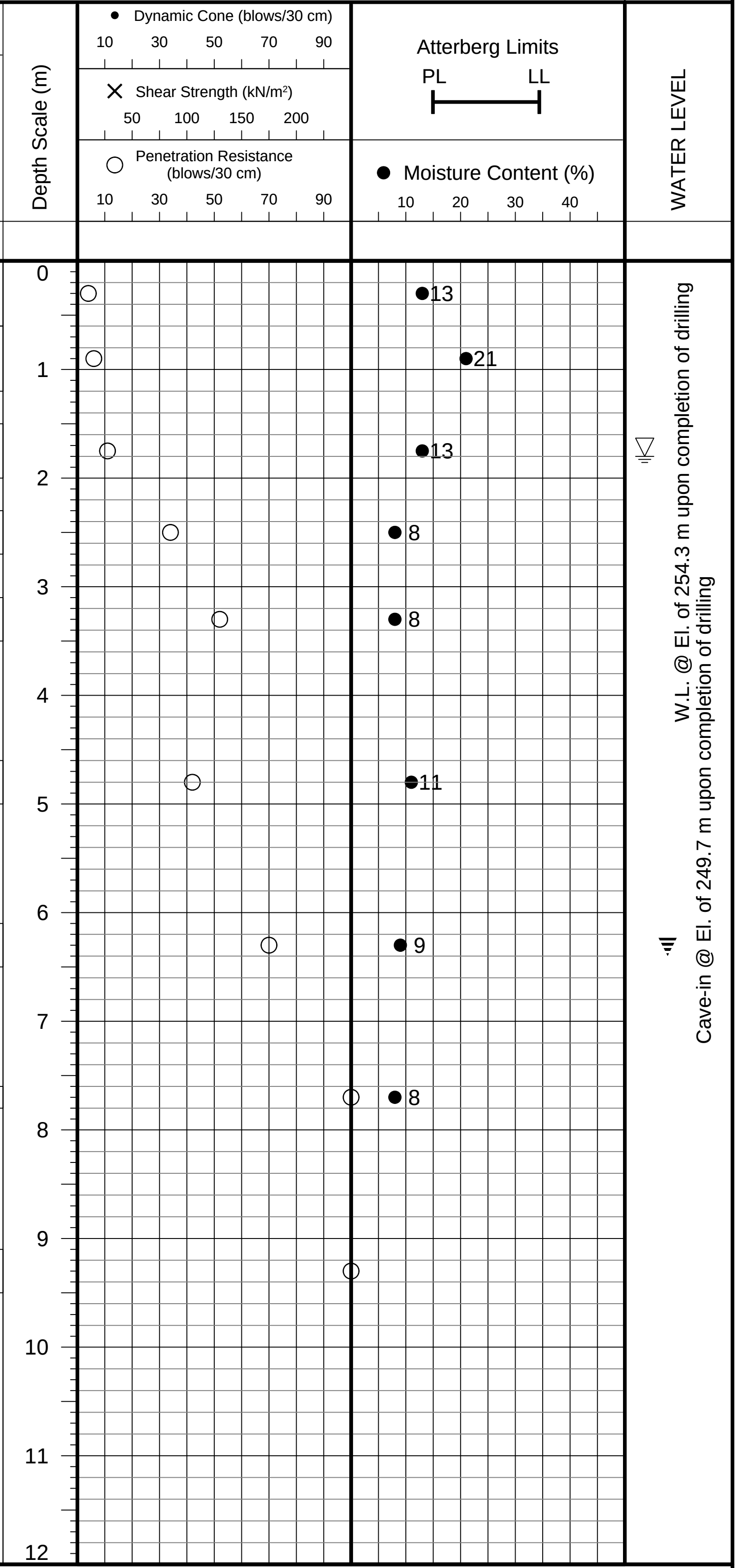
PROJECT DESCRIPTION: Proposed Condominium Development

METHOD OF BORING: Hollow Stem Auger

PROJECT LOCATION: 681 Yonge Street, City of Barrie

DRILLING DATE: April 18, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
256.1	Ground Surface			
0.0	**EARTH FILL** dark brown to brown, silty clay traces of sand and gravel occ. topsoil inclusions and organics	1 2	DO DO	4 6
254.6 1.5	Brown, compact to very dense **SILTY SAND TILL** a trace of gravel occ. cobbles and boulders — weathered —	3 4 5	DO DO DO	11 34 52
252.1 4.0	Brown, dense to very dense **SANDY SILT TILL** a trace of clay occ. sand seams, cobbles and boulders	6 7 8 9	DO DO DO DO	42 70 50/15 N/A
246.6 9.5	**END OF BOREHOLE**			



JOB NO.: 1903-S083

LOG OF BOREHOLE NO.: 5

FIGURE NO.: 5

PROJECT DESCRIPTION: Proposed Condominium Development

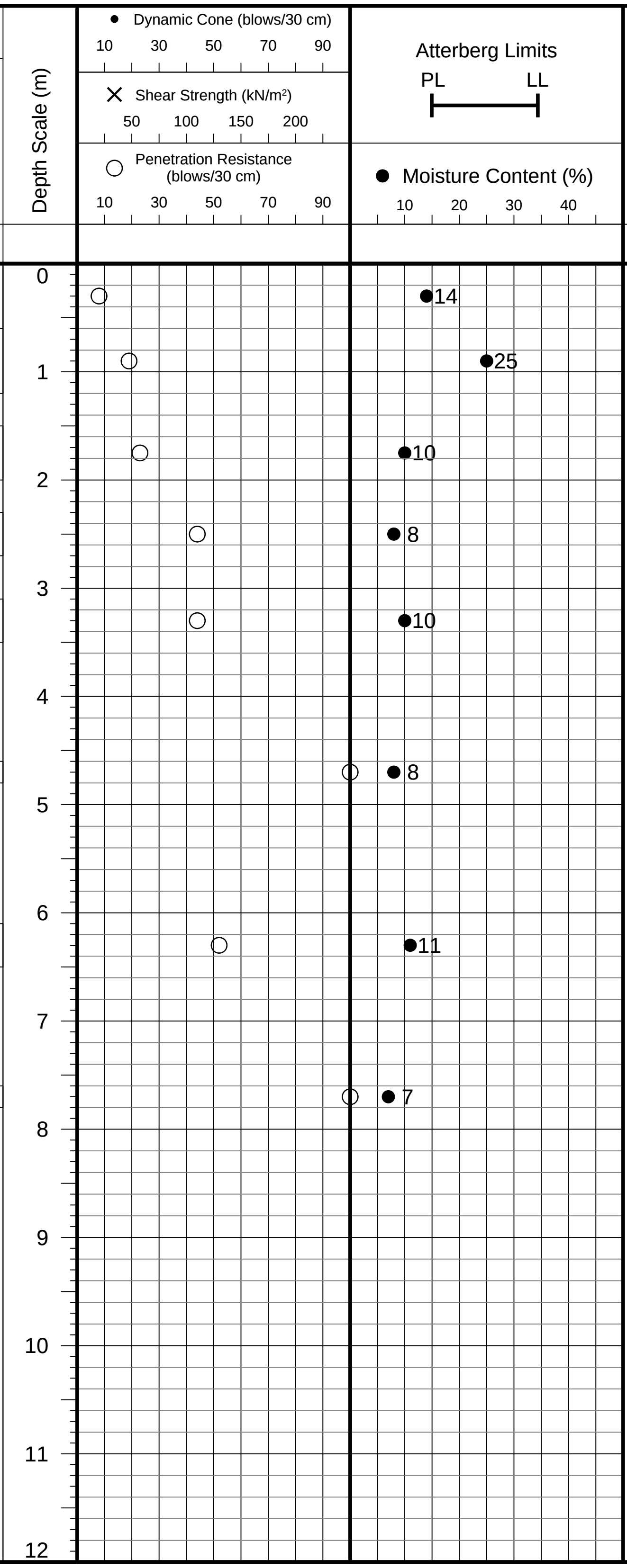
METHOD OF BORING: Hollow Stem Auger

PROJECT LOCATION: 681 Yonge Street, City of Barrie

DRILLING DATE: April 18, 2019

El. (m) Depth (m)	SOIL DESCRIPTION
256.9	Ground Surface
0.0	**EARTH FILL** dark brown, silty clay a trace of gravel occ. topsoil inclusions
255.5 1.4	Brown, compact to very dense **SANDY SILT TILL** traces of clay and gravel occ. sand seams, cobbles and boulders
249.9 7.0	Brown, very dense **SAND** fine to medium grained some gravel a trace of silt
249.1 7.8	**END OF BOREHOLE**

SAMPLES		
Number	Type	N-Value
1	DO	8
2	DO	19
3	DO	23
4	DO	44
5	DO	44
6	DO	50/15
7	DO	52
8	DO	50/15



WATER LEVEL

Cave-in @ El. of 250.8 m upon completion of drilling

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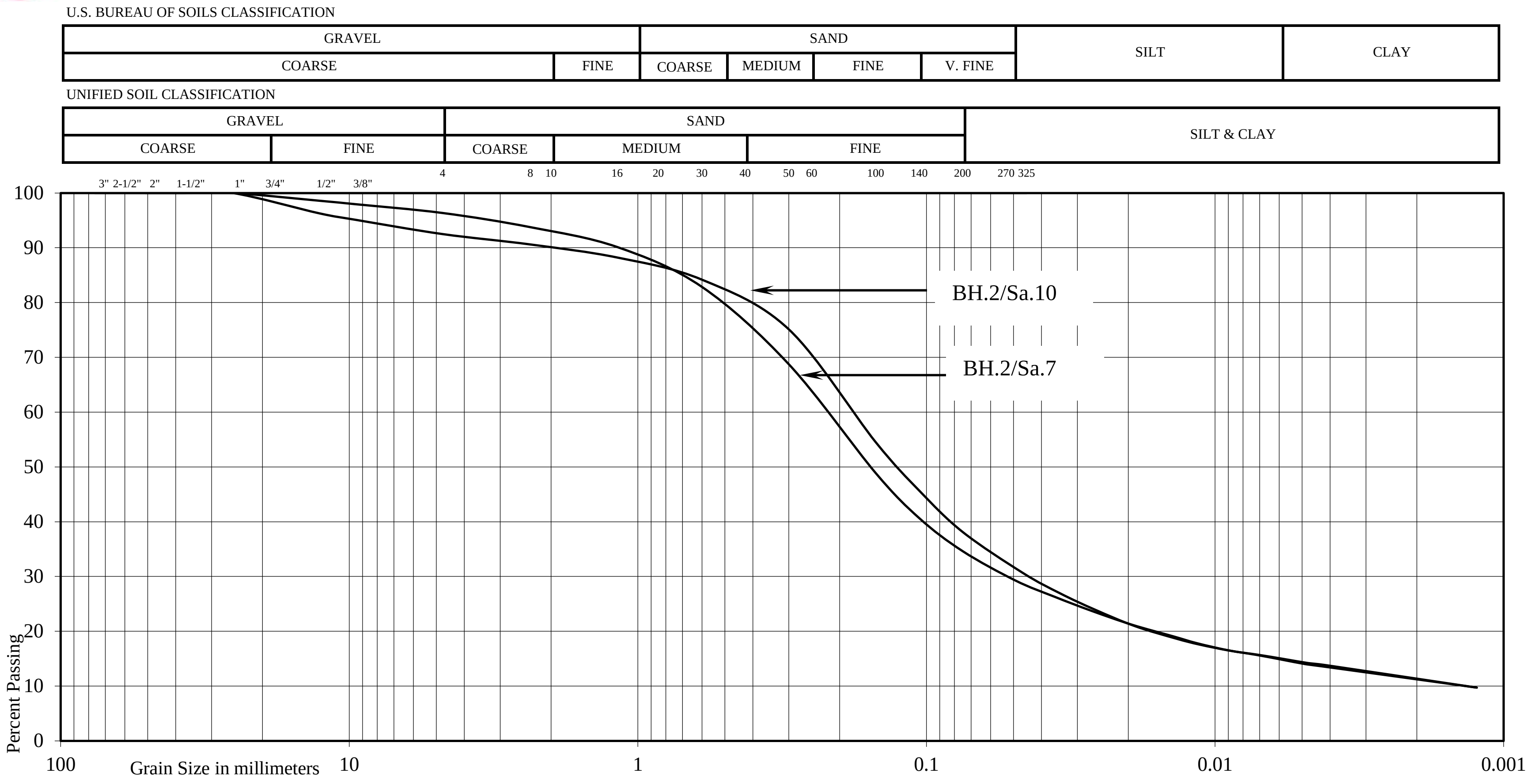
GRAIN SIZE DISTRIBUTION

Reference No: 1903-S083



Project: Proposed condominium Development

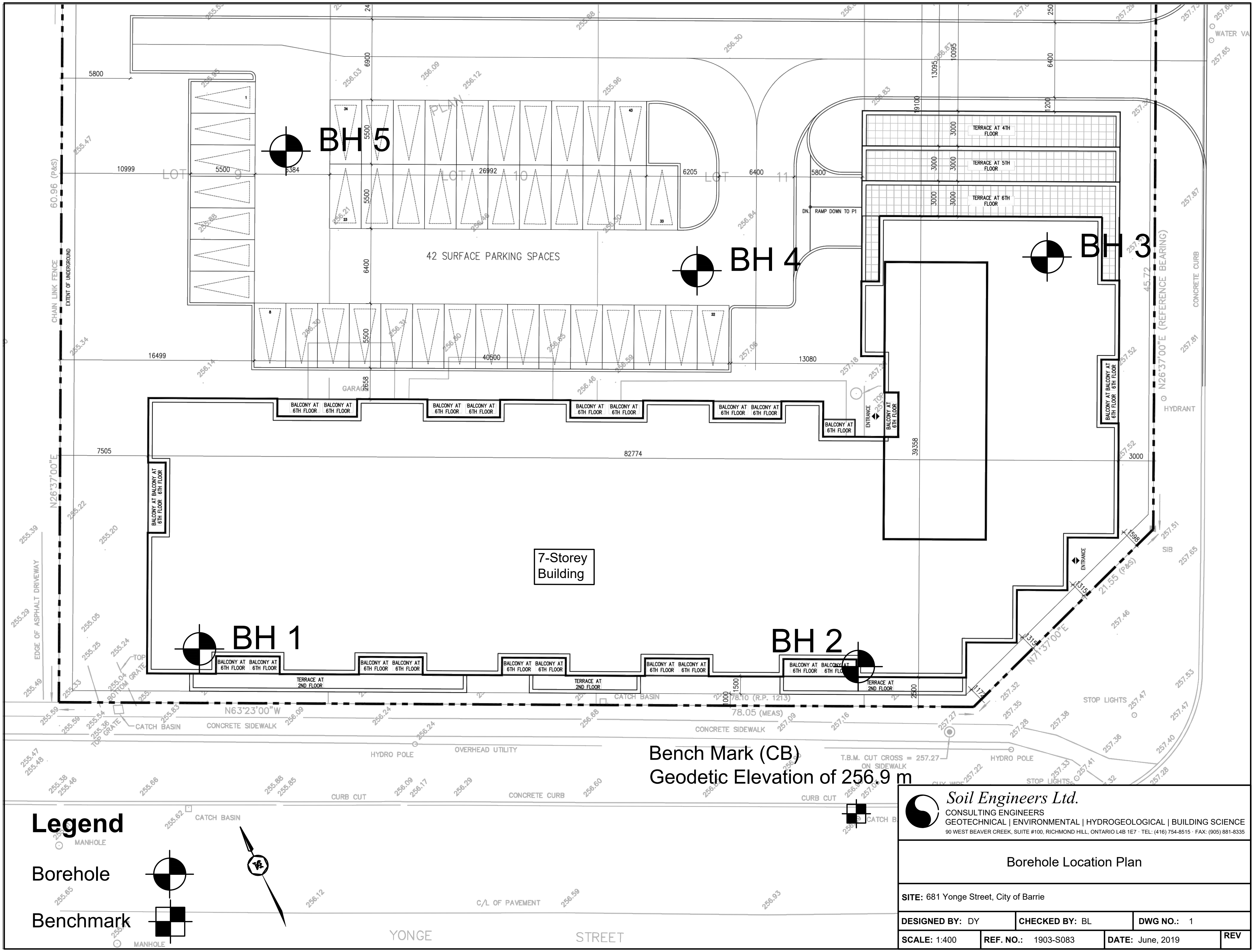
Location: 681 Yonge Street, City of Barrie

Borehole No:	2	2
Sample No:	7	10
Depth (m):	6.1	10.9
Elevation (m):	251.3	246.5

BH./Sa.	2/7	2/7
Liquid Limit (%) =	-	-
Plastic Limit (%) =	-	-
Plasticity Index (%) =	-	-
Moisture Content (%) =	7	11
Estimated Permeability (cm./sec.) =	10^{-6}	10^{-6}

Classification of Sample [& Group Symbol]: SILTY SAND TILL, some clay, a trace of gravel

Figure: 6



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90 WEST BEAVER CREEK, SUITE #100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

Borehole Location Plan

SITE: 681 Yonge Street, City of Barrie

DESIGNED BY: DY	**CHECKED BY:** BL	**DWG NO.:** 1
SCALE: 1:400	**REF. NO.:** 1903-S083	**DATE:** June, 2019	**REV**

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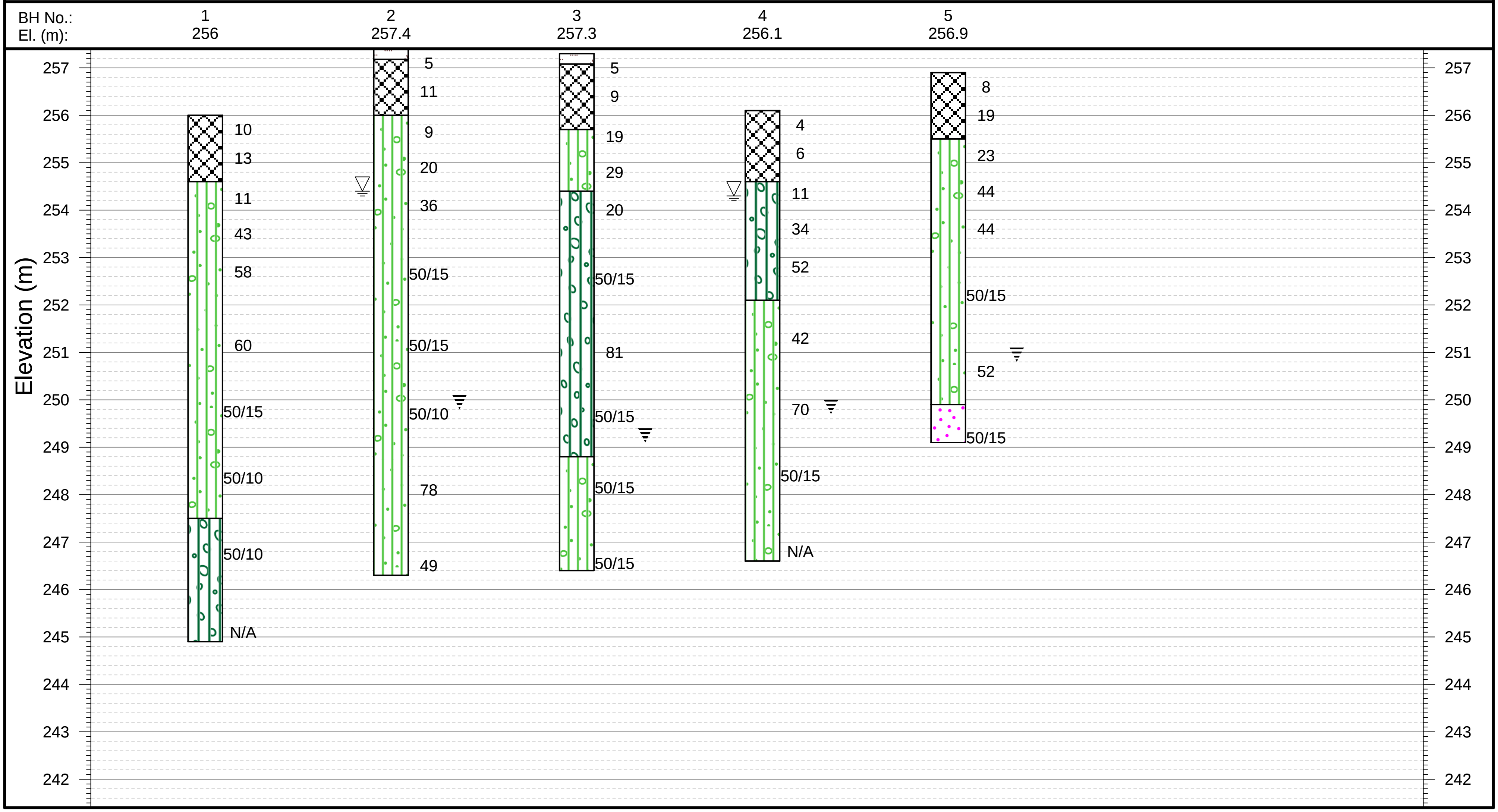
SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

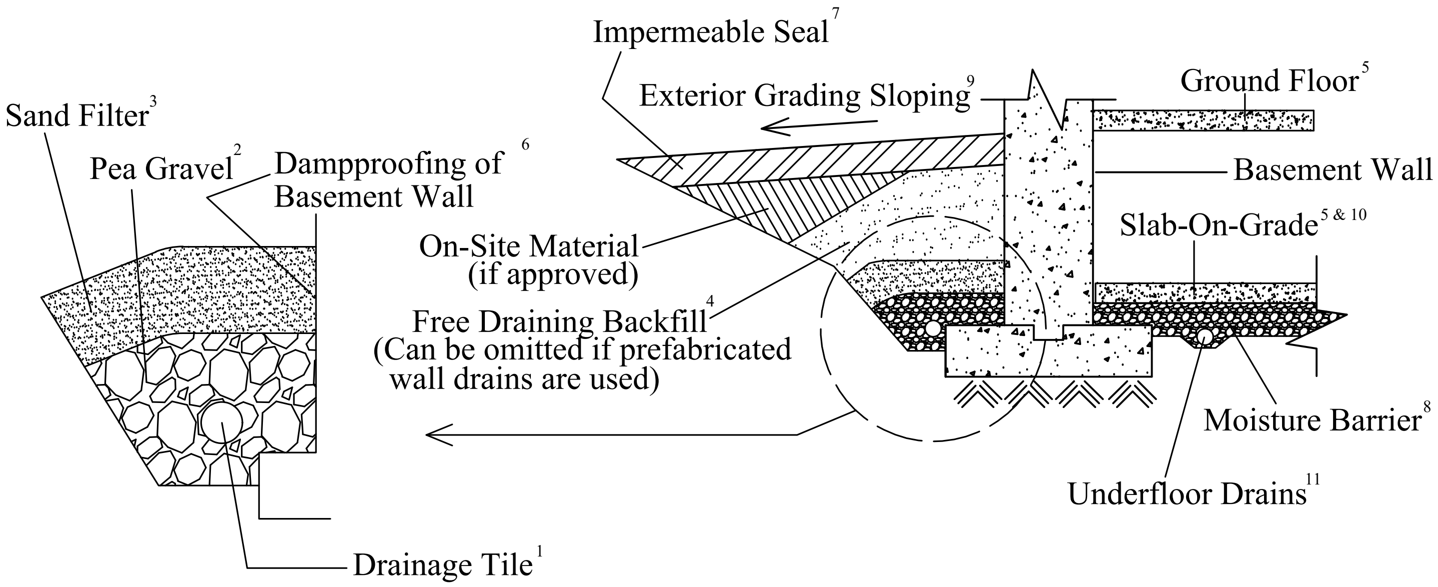
JOB NO.:	1903-S083
REPORT DATE:	June, 2019
PROJECT DESCRIPTION:	Proposed Condominium Development
PROJECT LOCATION:	681 Yonge Street, City of Barrie

LEGEND

TOPSOIL
SAND
SILTY SAND TILL
SANDY SILT TILL
FILL

CAVE-IN
WATER LEVEL (ON COMPLETION)





NOTES:

1. **Drainage tile**: consists of 100 mm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet. Invert to be at minimum of 150 mm (6") below underside of basement floor level.

2. **Pea gravel**: at 150 mm (6") on the top and sides of drain. If drain is not placed on concrete footing, provide 100 mm (4") of pea gravel below drain. The pea gravel may be replaced by 20 mm clear stone provided that the drain is covered by a porous geotextile membrane of Terrafix 270R or equivalent.

3. **Filter material**: consists of C.S.A. fine concrete aggregate. A minimum of 300 mm (12") on the top and sides of gravel. This may be replaced by an approved porous geotextile membrane of Terrafix 270R or equivalent.

4. **Free-draining backfill**: OPSS Granular 'B' or equivalent, compacted to 95% to 98% (maximum) Standard Proctor dry density. Do not compact closer than 1.8 m (6') from wall with heavy equipment. This may be replaced by on-site material if prefabricated wall drains (Miradrain) extending from the finished grade to the bottom of the basement wall are used.

5. **Do not backfill** until the wall is supported by the basement floor slab and ground floor framing, or adquate bracing.

6. **Dampproofing** of the basement wall is required before backfilling

7. **Impermeable backfill seal** of compacted clay, clayey silt or equivalent. If the original soil in the vicinity is a free-draining sand, the seal may be omitted.

8. **Moisture barrier**: 20-mm clear stone or compacted OPSS Granular 'A', or equivalent. The thickness of this layer should be 150 mm (6") minimum.

9. **Exterior Grade**: slope away from basement wall on all the sides of the building.

10. **Slab-On-Grade** should not be structurally connected to walls or foundations.

11. **Underfloor drains*** should be placed in parallel rows at 6 to 8 m (20'-25') centre, on 100 mm (4") of pea gravel with 150 mm (6") of pea gravel on top and sides. The invert should be at least 300 mm (12") below the underside of the floor slab. The drains should be connected to positive sumps or outlets. Do not connect the underfloor drains to the perimeter drains.

*Underfloor drains can be deleted where not required.

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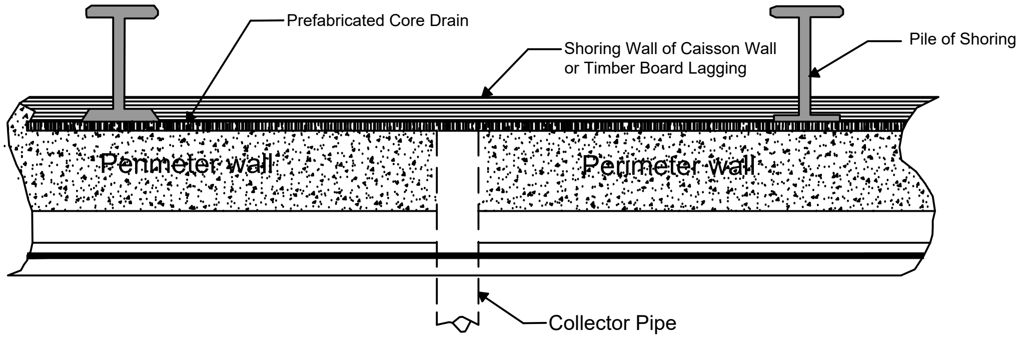
Permanent Perimeter Drainage System

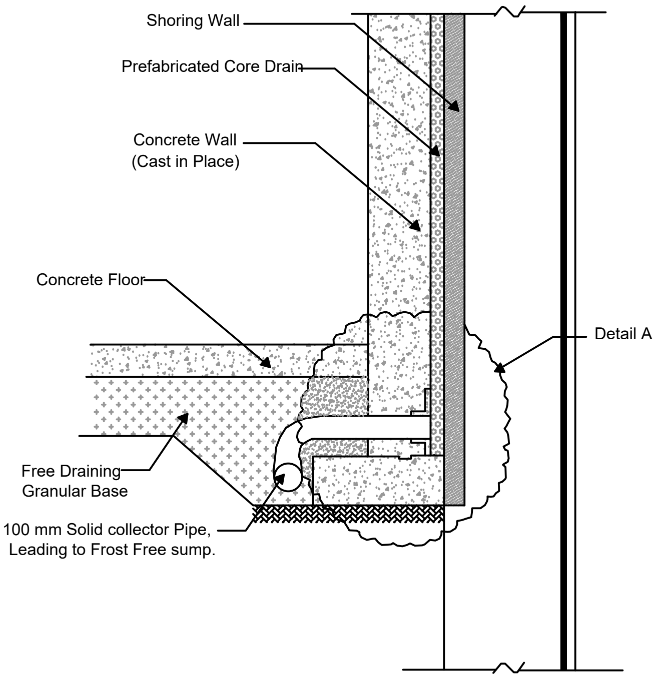
SITE 681 Yonge Stret, City of Barrie

DESIGNED BY D.Y. | **CHECKED BY** B.L. | **DWG NO.** 3

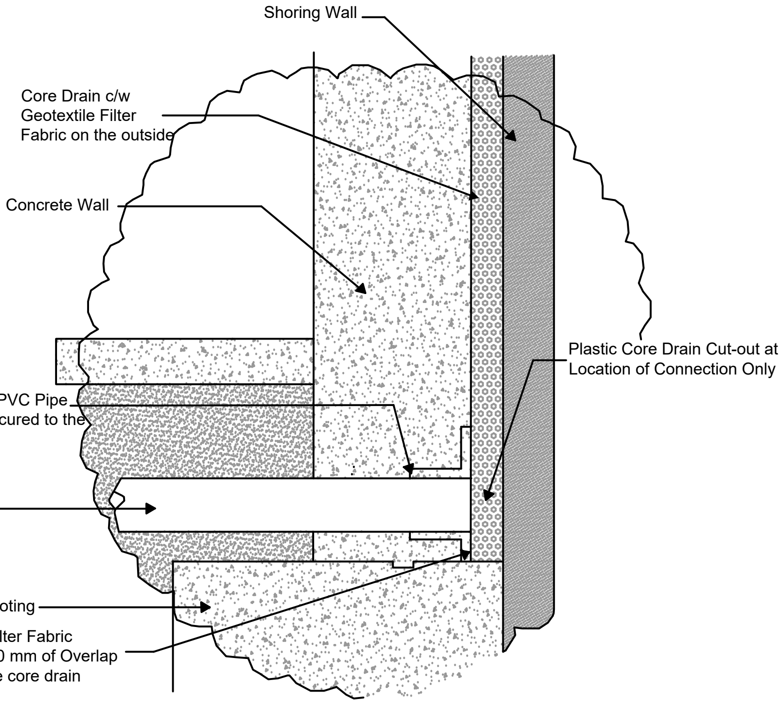
SCALE N.T.S. | **REF. NO.** 1903-S083 | **DATE** June, 2019 | **REV** -



PLAN



TYPICAL SECTION



DETAIL A

NOTES:

1. A continuous blanket of prefabricated drainage system, Miradrain 6000 or equivalent, should extend continuously from the top of footings to the ground surface.

2. All joints of the Miradrain should be taped. All openings above the concrete footing must be covered with filter fabric to prevent intrusion of fresh concrete into the core of the drain.

3. Backfill behind the lagging board must be free draining. Filter fabric or straw should be used to prevent loss of fines behind the lagging.

4. The perimeter drainage and any subfloor drainage systems must be kept separate.

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Permanent Perimeter Drainage System with Shoring

SITE: 681 Yonge Street, City of Barrie

DESIGNED BY: D.Y.	**CHECKED BY:** B.L.	**DWG NO.:** 4	
SCALE: N.T.S.	**REF. NO.:** 1903-S083	**DATE:** June, 2019	**REV** -

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FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 684-8522	FAX: (905) 725-1315	FAX: (905) 542-2769

APPENDIX

SHORING DESIGN

REFERENCE NO. 1903-S083

SHORING SYSTEM

Shoring will be required in an excavation to limit the horizontal and vertical movements of adjacent properties.

A shoring system consisting of soldier piles and lagging boards can be used in an excavation where slight movement in the adjacent properties is tolerable. In an area with close proximity of adjacent structure and the excavation will be extending below the foundation level where any movement in the adjacent properties is a concern, or in an excavation embedding into saturated sand or silt deposit, an interlocking caisson wall is more appropriate.

The design and construction of the shoring system should be carried out by a specialist designer and contractor experienced in this type of construction. All specifications for the design of the shoring system should be in accordance with the latest edition of the Canadian Foundation Engineering Manual (CFEM).

LATERAL EARTH PRESSURE

For single and multiple level supporting systems, the lateral earth pressure distributions on the shoring walls are shown on Drawing A1. The design soil parameters are provided in the geotechnical report.

The lateral earth pressure expressions do not include hydrostatic pressure buildup behind the shoring. If the wall is designed to be watertight or undrained, such as a caisson wall, the anticipated hydrostatic pressure must be included behind the structure.

PILE PENETRATION

The depth of pile support should be calculated from the following expressions:

$$R = 1.5 \, D \, K_p \, L^2 \gamma$$

where	R	= Ultimate load to be restrained	kN
	D	= Diameter of concrete filled hole	m
	K_p	= Passive resistance of soils below the level of excavation	
	L	= Embedment depth of the pile	m
	γ	= Unit weight of the soil	kN/m^3

The shoring system should be designed for a factor of safety of F = 2.

For anchor supported shoring system, the global factor of safety against sliding and overturning of the anchored block of soil must also be considered.

The steel soldier piles in the shoring system must be installed in pre-augured holes. The lower portion will have to be filled with 20 MPa (3000 psi) concrete to the excavation level. The upper portion of the pile within the excavation depth should be filled with lean mix concrete or non-shrinkable cementitious filler (U-fill).

LAGGING

The following thicknesses of lagging boards have been recommended in CFEM:

Thickness of Lagging	Maximum Spacing of Soldier Piles
50 mm (2 in)	1.5 m (5 ft)
75 mm (3 in)	2.5 m (8 ft)
100 mm (4 in)	3.0 m (10 ft)

Local experience has indicated that the lagging board thickness of 75 mm has been adequate for soldier pile spacing of 3 m for soil conditions similar to those encountered at the subject site. However, it is important to consider all local conditions, such as the duration of excavation, the weather likely to be encountered through the construction period, seasonal variations in the ground water and ice lensing causing frost heave and softening of soils in determining the lagging thickness. During winter months, the shoring should be covered with thermal blankets to prevent frost penetration behind the shoring system which may result in unacceptable movements.

During construction of shoring, all the spaces behind the lagging board must be filled with free-draining granular fill. If wet conditions are encountered, the space between the boards should be packed with a geotextile filter fabric or straw to prevent the loss of fine particles.

TIEBACK ANCHORS

The minimum spacing and the depths of the soil anchors should be as recommended in the CFEM.

All drilled holes for tieback anchors should be temporarily cased or lined to minimize the risk of caving. Systems involving high grout pressures should be avoided if working near other basements or buried services.

The tieback anchor lengths can be estimated using an adhesion values of 60 kPa. Full scale load tests should be carried out on the tieback anchors in each type of soils and at each level of anchor support at the site to confirm the design parameters and the adhesion values. The test anchors should be loaded in a pattern as described in CFEM, to 200% of the design load or until there is a significant increase in the pullout rate. In the latter case, the design load must be limited to 50% of the maximum load at which the pullout increases. Based on the results of the pullout test, it may be necessary to modify the anchor design of the production apnchors.

Each tieback anchor must be proof-loaded to 133% of the design load, and the anchor must be capable of sustaining this load for a minimum of 10 minutes without creep. The load may then be relaxed to 100% of the design and locked in. The higher the lock-in loads, the less will be the outward movement on the shoring wall after excavation.

RAKERS

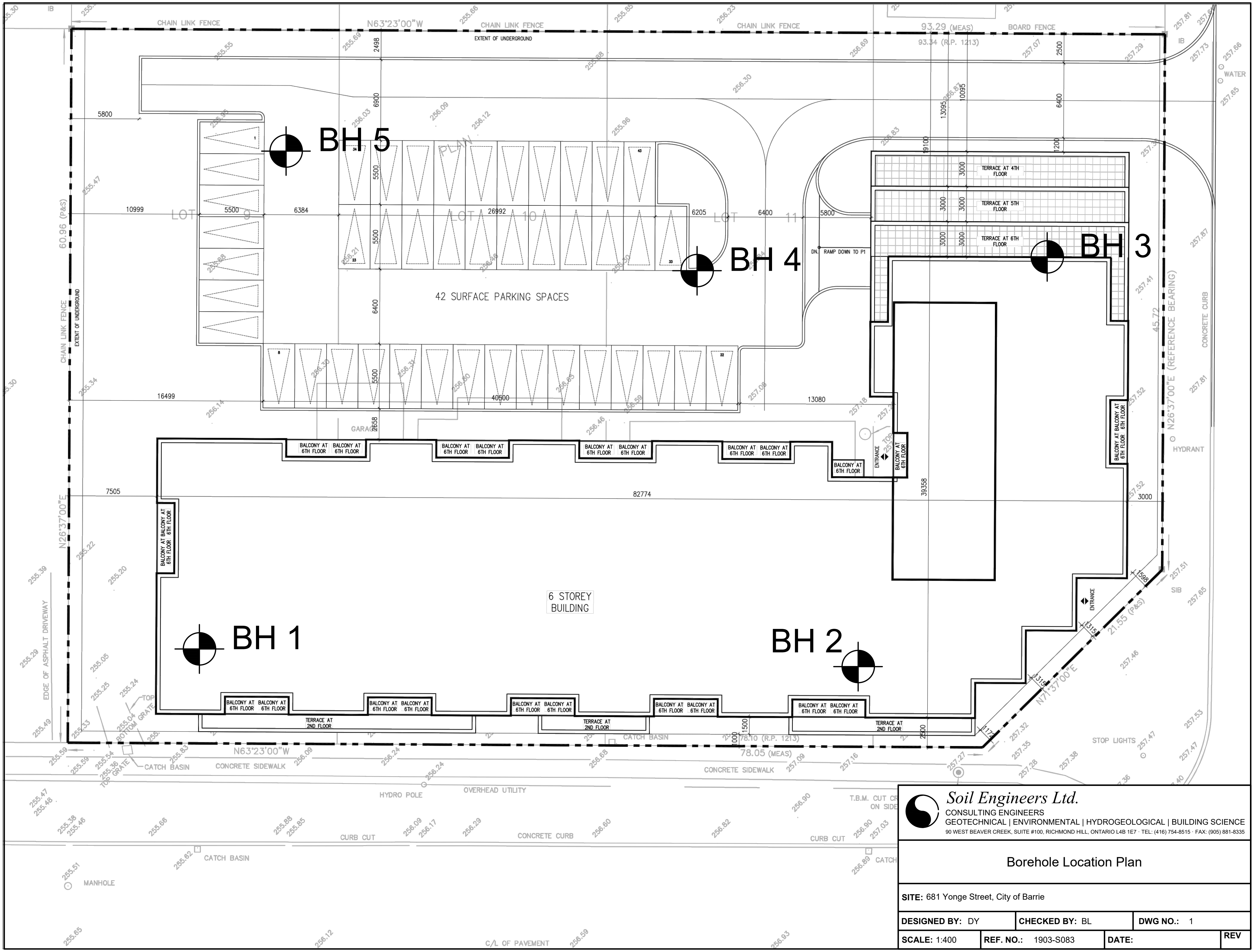
An alternative to tieback anchor support of the shoring is to use raker footings. Rakers inclining at an angle of 45º, founded in the native soil deposit below the bottom of excavation should be designed for the allowable bearing pressure of 30 kPa (0.6 k.s.f.).

The raker footings should be located outside the zone of influence of the buried portion of the soldier piles at a distance of not less than 1.5 of the length of embedment of the soldier pile.

To prevent undermining of the raker footing, no excavation should be made within two times the width of raker footing on the opposite side of the raker.

MONITORING OF PERFORMANCE

Close monitoring of the vertical and lateral movement of the shoring system, by inclinometers or by survey on targets, should be carried out at the site. Extra bracing or support may be required if any movement is found excessive. The contractor should maintain the shoring to ensure any movement is within the design limit.



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Borehole Location Plan

SITE: 681 Yonge Street, City of Barrie

DESIGNED BY: DY **CHECKED BY:** BL **DWG NO.:** 1

SCALE: 1:400 **REF. NO.:** 1903-S083 **DATE:** **REV**