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A REPORT TO
SEAN MASON HOMES (ESSA RD) INC.

A GEOTECHNICAL INVESTIGATION FOR PROPOSED RESIDENTIAL BUILDING WITH UNDERGROUND PARKING

405 ESSA ROAD

CITY OF BARRIE

REFERENCE NO. 2007-S085

JULY 2021

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1.0 INTRODUCTION

In accordance with written authorization from Mr. Sean Mason of Sean Mason Homes (Essa Rd) Inc., dated July 16, 2020, a geotechnical investigation was carried out at 405 Essa Road, 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 residential building with underground parking. The geotechnical findings and resulting recommendations are presented in this Report.

It should be noted that Soil Engineers Ltd. has previously completed a borehole investigation at this site, which included 3 boreholes extending to depths of 3.4 m and 6.4 m. The findings, as presented in our Geotechnical Investigation Report, Reference No. 1710-S170, dated February 2018, have been incorporated into 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 site is located on the north side of Essa Road, east of Ferndale Drive South, in the City of Barrie. At the time of the investigation, the site was a vacant lot with trees and weed growth. We understand that a single storey house, previously located on site, has been demolished/removed. The northwest portion was generally cleared of vegetation, with trees at the perimeter. The existing ground surface is relatively flat.

It is understood that the proposed development will consist of a mid-rise residential building and attached townhouse units, with an adjoined underground parking garage of 1 to 2 levels.

3.0 FIELD WORK

The current field work, consisting of 2 boreholes to a depth of 8.1 m, was performed on July 29, 2020. In order to distinguish these boreholes from the previous investigation, the present boreholes are numbered in the 100-series. The previous investigation of 3 boreholes to depths of 3.4 m and 6.4 m was performed on November 20, 2017. The borehole locations are shown on Drawing No. 1.

The boreholes were advanced at intervals to the sampling depths by track-mounted, continuous-flight power-auger machines 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 non-cohesive 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 Boreholes 101 and 102 were determined in relation to a reference point, “Double Catch Basin”, located on the west side of Essa Road, approximately 60 m south of the driveway entrance into the site. It has a reference elevation of 311.2 m; however, this can be confirmed by a licensed surveyor. The ground elevation of previous Boreholes 1, 2 and 3 were determined with reference to a manhole located on the northbound lane on Essa Road, having a geodetic elevation of 311.59 m. The locations of the reference benchmarks are shown on Drawing No. 1.

4.0 **SUBSURFACE CONDITIONS**

The borehole investigations have revealed that beneath a topsoil layer or granular surface at some boreholes, with a layer of earth fill at one borehole, the site is underlain by strata of sand, silty fine sand and/or silty sand till at various locations and depths.

Detailed descriptions of the encountered subsurface conditions of the current boreholes are presented on the Borehole Logs, comprising Figures 1 and 2; the logs of the previous boreholes are included in Appendix ‘A’. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing No. 2. The engineering properties of the disclosed soils are discussed herein.

4.1 **Topsoil** (Boreholes 2, 3, 101 and 102)

The revealed topsoil layer at Borehole 102 is approximately 15 cm thick, and was 20 cm and 30 cm thick at Boreholes 2 and 3, respectively. In addition, a layer of topsoil was encountered beneath the earth fill at Borehole 101.

4.2 **Granular Fill** (Borehole 1)

A layer of granular fill, approximately 100 mm thick, was encountered at the ground surface in the area of Borehole 1; it consisted of medium to coarse grained sand with occasional gravel and crushed stone.

4.3 **Earth Fill** (Borehole 101)

A layer of earth fill was encountered at Borehole 101, at the ground surface, and extends to a depth of 0.5± m from the prevailing ground surface; it consists of silty sand with organic inclusions.

The obtained 'N' value is 21 blows per 30 cm of penetration, indicating the earth fill was placed with compactive effort.

The natural water content of the sample was determined and the result is plotted on the Borehole Log; the value is 9%, indicating that the earth fill is in a moist condition.

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.

4.4 **Sand** (All Boreholes)

The sand deposit was interstratified in the upper to mid zone of the revealed soil stratigraphy; it is fine grained and contains a trace to some silt with occasional gravel. The layered structure shows that the sand is a lacustrine deposit. The sand near the ground surface has been loosened by the weathering process.

The obtained 'N' values range 3 blows per 30 cm of penetration to 50 per 15 cm, with a median of 22 per 30 cm, indicating the relative density of the sand is very loose to very dense, being generally compact. The very loose to loose sand is restricted to the weathered zone near the ground surface.

The natural water content of the samples was determined and the results are plotted on the Borehole Logs; the values range from 2% and 16%, with a median of 7%, indicating damp to wet conditions. The wet sand generally occurs within the weathered zone near the ground surface.

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

- Moderately low frost susceptibility.
- High water erodibility; susceptible to migration through small openings under seepage pressure.
- 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

- The shear strength is derived from internal friction and is soil density dependent.
- In excavation, the sand will slough, run with seepage and boil under a piezometric head of 0.4 m.
- A fair pavement-supportive material, with an estimated California Bearing Ratio (CBR) value of 15%.
- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 6000 ohm·cm.

4.5 **Silty Fine Sand** (Boreholes 1, 101 and 102)

The silty fine sand was contacted in the mid to lower zone of the revealed stratigraphy beneath the sand and/or till stratum; it contains a trace of clay with clay or silt layers in places. The sorted structure indicates that the silty fine sand is a glaciolacustrine deposit.

The obtained 'N' values range from 35 blows per 30 cm of penetration to 50 per 15 cm, with a median of 47 per 30 cm, indicating the relative density of the silty fine sand is dense to very dense, being generally dense.

The natural water content of the samples was determined and the results are plotted on the Borehole Logs; the values range from 10% to 19%, with a median of 12%, indicating very moist to wet conditions. The wet samples are water bearing and displayed dilatancy.

A grain size analysis was performed on 1 representative silty fine sand sample; the result is plotted on Figure 3.

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

- High frost susceptibility and high soil-adfreezing potential.
- High water erodibility; it is susceptible to migration through small openings under seepage pressure.
- A soil of high capillarity and water retention capacity.
- Relatively pervious, with an estimated coefficient of permeability of 10^{-3} to 10^{-4} cm/sec, and runoff coefficients of:

Slope	
0% - 2%	0.04 to 0.07
2% - 6%	0.09 to 0.12
6% +	0.13 to 0.18

- The shear strength is derived from internal friction and is soil density dependent. Due to its dilatancy, the strength of the wet sand 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 excavation, the sand will slough, run slowly with water seepage, and boil under a piezometric head of 0.4 m.
- A poor pavement-supportive material, with an estimated CBR value of 5%.
- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 5500 to 6000 ohm·cm.

4.6 **Silty Sand Till** (Boreholes 2, 3, 101 and 102)

The silty sand till was encountered below the sand deposit and overlays silty fine sand in the deeper boreholes. It consists of a random mixture of soils; the particle sizes range from clay to gravel, with the sand and silt fraction exerting the dominant influence on the soil properties. The till is heterogeneous and amorphous, with occasional sand seams and layers, cobbles and boulders, showing it is a glacial deposit which has been partially reworked by the past glaciation. The till within the top 1.8± to 2.3± m from the prevailing ground surface is permeated with fissures and fractured by the weathering process.

The obtained 'N' values range from 6 blows per 30 cm of penetration to 50 per 15 cm, with a median of 46 per 30 cm, indicating that the relative density of the till is loose to very dense, being generally dense.

The natural water content of the samples was determined and the results are plotted on the Borehole Logs; the values range from 5% to 12%, with a median of 8%, indicating damp to moist conditions.

Grain size analyses were performed on 3 representative samples of the silty sand till; the results are plotted on Figure 4 and in Appendix 'A'.

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

- High frost susceptibility and moderately low water erodibility.
- Relatively low permeability, with an estimated coefficient of permeability of 10^{-4} to 10^{-5} cm/sec, and runoff coefficients of:

Slope	
0% - 2%	0.07 to 0.11
2% - 6%	0.12 to 0.16
6% +	0.18 to 0.23

- The shear strength is primarily derived from the internal friction and is augmented by cementation.
- The till is relatively stable in steep cuts; however, under prolonged exposure, localized sheet collapse may occur, particularly in the zone where sand and silt layers are prevalent.
- A fair pavement-supportive material, with an estimated CBR value of 10%.
- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 5000 ohm·cm.

5.0 GROUNDWATER CONDITIONS

The boreholes were checked for the presence of groundwater and the occurrence of cave-in upon their completion. Groundwater was recorded at depths of 7.9± m and 8.0± m at Boreholes 101 and 102, respectively, upon completion of the field work. The other boreholes, which were relatively shallow, remained dry and open during and upon completion.

With reference to the water content profile of the soil samples and the field observations, the groundwater recorded in the open boreholes may represent the groundwater regime and it will fluctuate with the seasons.

The groundwater yield in excavation above the groundwater regime is expected to be slight in quantity, which can be controlled by conventional pumping from sumps, where necessary. Any excavation extending into the groundwater regime will require extensive dewatering.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigations have revealed that beneath a topsoil layer or granular surface at some boreholes, with a layer of earth fill at one borehole, the site is underlain by strata of very loose to very dense, generally compact sand, dense to very dense, generally dense silty fine sand and/or loose to very dense, generally dense silty sand till at various locations and depths. The soil within the top 0.8± to 2.3± m from the ground surface has been weathered.

Groundwater was recorded in the deeper boreholes at depths of 7.9± m and 8.0± m below the prevailing ground surface. The groundwater level will fluctuate with the seasons.

The proposed development will consist of a mid-rise residential building and attached townhouse units, with an adjoined underground parking garage of 1 to 2 levels.
The geotechnical findings which warrant special consideration are presented below:

1. Excavation for the 1-level underground parking will extend to an approximate depth of 3 to 4 m into the dense to very dense sands or till.
2. Excavation for the 2-level underground parking in the west portion of the property will extend to an approximate depth of 6 or 7 m into the dense to very dense sands or till.
3. Groundwater encountered in the open boreholes, representing the groundwater regime, was recorded in the deeper boreholes at depths of 7.9± m and 8.0± m from the existing grade. The groundwater yield in excavation above the groundwater regime is expected to be slight in quantity, which can be controlled by conventional pumping from sumps, where necessary.
4. The proposed structures can be constructed on conventional footings or a raft foundation at the founding level. Please note that the final design of foundation should be reviewed by the geotechnical engineer.
5. Where space is not sufficient for a safe backing slope, the excavation should be supported by a braced shoring system.
6. Due to the presence of adjacent buildings, the foundation details of the adjacent structures must be investigated and incorporated into the excavation, design and construction of the underground structure. It is recommended that a pre-construction survey and a monitoring program be carried out for all adjacent structures in order to verify any potential future liability claims.

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 the subsurface variances become apparent during construction, a geotechnical engineer must be consulted to determine whether the following recommendations require revision.

6.1 Foundations

The proposed development will consist of a mid-rise residential building and attached townhouse units, with an adjoined 1- to 2-level underground parking garage. The excavation is expected to extend into the dense to very dense stratum.

For the design of structure with 1-level and 2-levels underground parking, the recommended bearing pressures for conventional footing design are presented below:

- Maximum Soil Bearing Pressure at Serviceability Limit State (SLS) = 400 kPa
- Factored Ultimate Bearing Pressure at Ultimate Limit State (ULS) = 600 kPa

For the structure with 2-level underground parking, excavation will extend to an approximate depth of 6 or 7 m into the dense to very dense sands or till.

The construction of conventional footings will require the underground structure to be drained into the subdrain system as described in Section 6.2. If the groundwater level is relatively higher than the parking deck or any passive drainage of groundwater is not feasible, the underground structure will have to be waterproofed. 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.

Underground structures to be waterproofed should be constructed on a raft foundation to resist the hydrostatic pressure. The design bearing pressures for a raft foundation are provided below:

- Maximum Soil Bearing Pressure at SLS = 350 kPa
- Factored Ultimate Bearing Pressure at ULS = 500 kPa

A Modulus of Subgrade Reaction (k_s) of 25 MPa/m can be used for the design of the raft foundation.

The total and differential settlements of footings designed for the bearing pressures at SLS are estimated to be 25 mm and 20 mm, respectively.

Foundations exposed to weathering, or in unheated areas, should have at least 1.5 m of earth cover for protection against frost action. For the unheated underground parking structure, having the entrance door closed most of the time, the earth cover can be reduced to 0.8 m for the perimeter walls and 1.2 m for the interior walls and columns, except in the area in close proximity to ventilation shafts and the door entrances.

The building foundation should be inspected by a geotechnical engineer or a senior geotechnical technician to ensure that the revealed conditions are compatible with the foundation design requirements.

A mud slab of lean mix concrete, 8 to 10 cm in thickness, will be required on the founding subgrade to provide a working platform for the workers, if the excavation will be left open for construction or installation of reinforcement in the raft foundation.

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

Please note that the final design of foundation should be reviewed by the geotechnical engineer.

6.2 **Underground Parking**

In conventional design, the perimeter walls of the underground garage should be dampproofed and provided with a perimeter subdrain encased in a fabric filter at the wall base. Prefabricated drainage board, such as Miradrain 6000 or equivalent, must be provided between the shoring wall and the cast-in-place foundation wall, or adjacent to the foundation wall in open excavation, as shown on Drawing Nos. 3 and 4, respectively.

The underground parking floor slab should be constructed on a granular bedding, consisting of 20-mm Crusher-Run Limestone, or equivalent, compacted to its maximum SPDD. Where the subgrade is wet or consisting of wet sand, a subfloor drainage system, consisting of 100-mm filter-sleeved weepers, should be installed in a grid pattern, with a centre-to-centre spacing of 5.0 to 6.0 m, and must be connected to a positive outlet or sump pit for discharge. In addition, vapour barrier of 10 mil polyethylene sheet should be placed above the granular bedding to alleviate wetting of the garage floor due to moisture up-filtration. A typical design of the bedding and the underfloor weepers is provided in Drawing No. 5.

If the Municipality does not allow any discharge of the subsurface water into the sewer system, a separate storage cistern should be provided or, otherwise, the entire underground structure will have to be waterproofed. In this case, the building will have to be founded on a raft foundation, and designed for the full depth of hydrostatic pressure on the foundation walls and below the foundation. The concrete slab of the parking deck will be poured on a granular fill above the raft where the utilities and service pipes will be laid.

The underground structure should be designed to sustain a lateral earth pressure calculated using the soil parameters stated in Section 6.7. Any applicable surcharge loads adjacent to the proposed building must also be considered in the design of the underground structure.

At the garage entrance, the subgrade should be properly insulated, or the subgrade material should be replaced with 1.5 m of non-frost-susceptible granular material and 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 to prevent ponding of water in the areas adjacent to the building.

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

6.3 Underground Services

The subgrade for the underground services should consist of sound natural soils or properly compacted organic-free earth fill. Where organic earth fill or badly weathered soil is encountered, it should be subexcavated and replaced with bedding material compacted to at least 95% or + SPDD.

A Class ‘B’ bedding is recommended for the underground services construction. The bedding material should consist of compacted 20-mm Crusher-Run Limestone, or equivalent. In the areas of saturated soil or where extensive dewatering is required, a Class ‘A’ concrete bedding will be required. The pipe joins into manholes should be leak-proof, or the joints should be wrapped with a waterproof membrane, to prevent subgrade upfiltration through the joints.

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

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

All metal fittings for the underground services should be protected against soil corrosion. The in situ 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 pipe alignment at the time of construction.

6.4 Backfilling in Trenches and Excavated Areas

The backfill in trenches and excavated areas should be organic free material compacted to at least 95% SPDD and increased to 98% or + SPDD below the floor slab. In the zone within 1.0 m below the pavement subgrade, the materials should be compacted with the water content 2% to 3% drier than the optimum, and the compaction should be increased to at least 98% SPDD. This is to provide the required stiffness for pavement construction.

In normal construction practice, the problem areas of settlement largely occur adjacent to manholes, catch basins, services crossings, foundation walls and columns; it is recommended that a granular backfill should be used in confined spaces.

Narrow trenches for services crossings should be cut at 2 horizontal (H):1 vertical (V), or flatter, 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 soils have a water content on the dry side of the optimum, it would be impossible to wet the soils due to the freezing condition, rendering difficulties in obtaining uniform and proper compaction. Furthermore, the freezing condition will prevent wetting of the backfill when it is required, such as in a narrow vertical trench section, or when the trench box is removed. The above will invariably cause backfill settlement that may become evident within 1 to several years, depending on the depth of the trench which has been backfilled.

- In areas where the 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 final surfacing of the new pavement.
- In deep trench backfill, one must be aware that future settlement is to be expected, unless the side of the cut is flattened to at least 1.5+H:1V, 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 98% SPDD, 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 or non shrinkable fill, and the compaction must be carried out diligently 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.

6.5 Sidewalks, Interlocking Stone Pavement and Landscaping

Due to the high frost susceptibility of some of the underlying soils, heaving of the sidewalk and interlocking pavement structures is expected to occur during the cold seasons. On-grade sidewalk and pavement structures must be designed to tolerate ground movement due to frost heaving.

In areas which are sensitive to frost-induced ground movement, such as the sidewalk and barrier-free ramp in front of building entrances, they can be constructed on a free-draining, non-frost-susceptible granular material such as Granular ‘B’, extending to a depth of 1.5 m, and provided with weeper subdrains at the base draining towards manholes or catch basins.

Alternatively, the sidewalk and barrier-free ramp subgrade can be insulated with 50-mm Styrofoam, with free-draining granular material for the ramp construction above grade only.

6.6 Pavement Design

Where the pavement is to be built on structural slabs, such as the rooftop of the underground garage, 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 steel reinforcing bars against brine corrosion. The recommended pavement structure to be placed on the underground garage rooftop is presented in Table 1.

Table 1 - Pavement Design (Roof of Underground Garage)

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	50	HL-8
Granular Base	200	OPSS Granular ‘A’ or equivalent
Granular Sub-base	100	Free-draining Sand Fill

For the on-grade portion of pavement and driveway access from Essa Road, the recommended pavement structure is given in Table 2.

Table 2 - Pavement Design (On-Grade Pavement)

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	50	HL-8
Granular Base	150	OPSS Granular ‘A’ or equivalent
Granular Sub-base	350	OPSS Granular ‘B’ or equivalent

In preparation of the pavement subgrade, the final subgrade surface must be proof-rolled. The earth fill, or any weathered or loose soil must be subexcavated, sorted free of any deleterious materials, aerated and properly compacted to the density as specified in Section 6.4.

The granular bases should be compacted to 100% SPDD.

Along the perimeter where surface 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 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.

6.7 Soil Parameters

The recommended soil parameters for the project design are given in Table 3.

Table 3 - Soil Parameters

Unit Weight and Bulk Factor

	Unit Weight (kN/m^3)		Estimated Bulk Factor	
	Bulk	**Submerged**	**Loose**	**Compacted**
Existing Earth Fill	20.5	10.8	1.20	0.98
Sand	20.0	10.8	1.25	1.00
Silty Fine Sand	20.5	10.5	1.20	0.98
Silty Sand Till	22.5	12.5	1.33	1.05

Lateral Earth Pressure Coefficients

	Active K_a	At Rest K_0	Passive K_p
Compacted Earth Fill	0.40	0.55	2.50
Native Sands and Till	0.32	0.48	3.12

Coefficients of Friction

Between Concrete and Granular Base	0.60
Between Concrete and Sound Natural Soils	0.40

6.8 Excavation

Where excavation is to be carried out close to any existing underground structure or services, one must be aware that the previous backfill is amorphous in structure and is susceptible to sloughing and sudden side collapse. Extreme caution must be exercised and test pits should be used to evaluate the safety of such excavation. The existing services must be properly secured, where necessary.

Excavation should be carried out in accordance with Ontario Regulation 213/91. For excavation purposes, the types of soils are classified in Table 4.

Table 4 - Classification of Soils for Excavation

Material	Type
Sound natural Till	2
Earth Fill, weathered Soil and dewatered Sand	3
Saturated Sand	4

In areas where a safe backing slope is not possible, the excavation has to be supported by shoring. The overburden load and the surcharge from adjacent structures should be included in the design of the shoring. The design parameters and our recommendations for shoring are provided in Appendix 'B'.

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.

Groundwater encountered in the open boreholes, representing the groundwater regime, was recorded in the deeper boreholes at depths of 7.9± m and 8.0± m from the existing grade. The groundwater yield in excavation above the groundwater regime is expected to be slight in quantity, which can be controlled by conventional pumping from sumps, where necessary. Any excavation extending into the groundwater regime will require extensive dewatering. This should be assessed by test pumping prior to the project construction when the intended bottom of excavation is determined. In order to provide a stable subgrade for the services or foundation construction, the groundwater should be depressed at least 1.0 m below the subgrade level.

6.9 **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.

Due to the presence of nearby 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 a monitoring program be carried out for all adjacent structures in order to verify any potential future liability claims.

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

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the accounts of Sean Mason Homes (Essa Rd) Inc., and for review by the designated consultants and government agencies. Use of the report is subject to the conditions and limitations of the contractual agreement.

The material in the report reflects the judgement of Mumta Mistry, B.A.Sc., and Bennett Sun, P.Eng., in light of the information available to it at the time of preparation. Any use which a Third Party makes of this report, or any reliance on decisions to be made based on it, are the responsibility of such Third Parties. Soil Engineers Ltd. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

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July 23, 2021
PROVINCE OF ONTARIO

LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

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

SAMPLE TYPES

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

PENETRATION RESISTANCE

Dynamic Cone Penetration Resistance:

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

Plotted as '—•—'

Standard Penetration Resistance or 'N' Value:

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

Plotted as '○'

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

SOIL DESCRIPTION

Cohesionless Soils:

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

Cohesive Soils:

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

Method of Determination of Undrained Shear Strength of Cohesive Soils:

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

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

METRIC CONVERSION FACTORS

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

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JOB NO.: 2007-S085

LOG OF BOREHOLE NO.: 101

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed Residential Building with Underground Parking

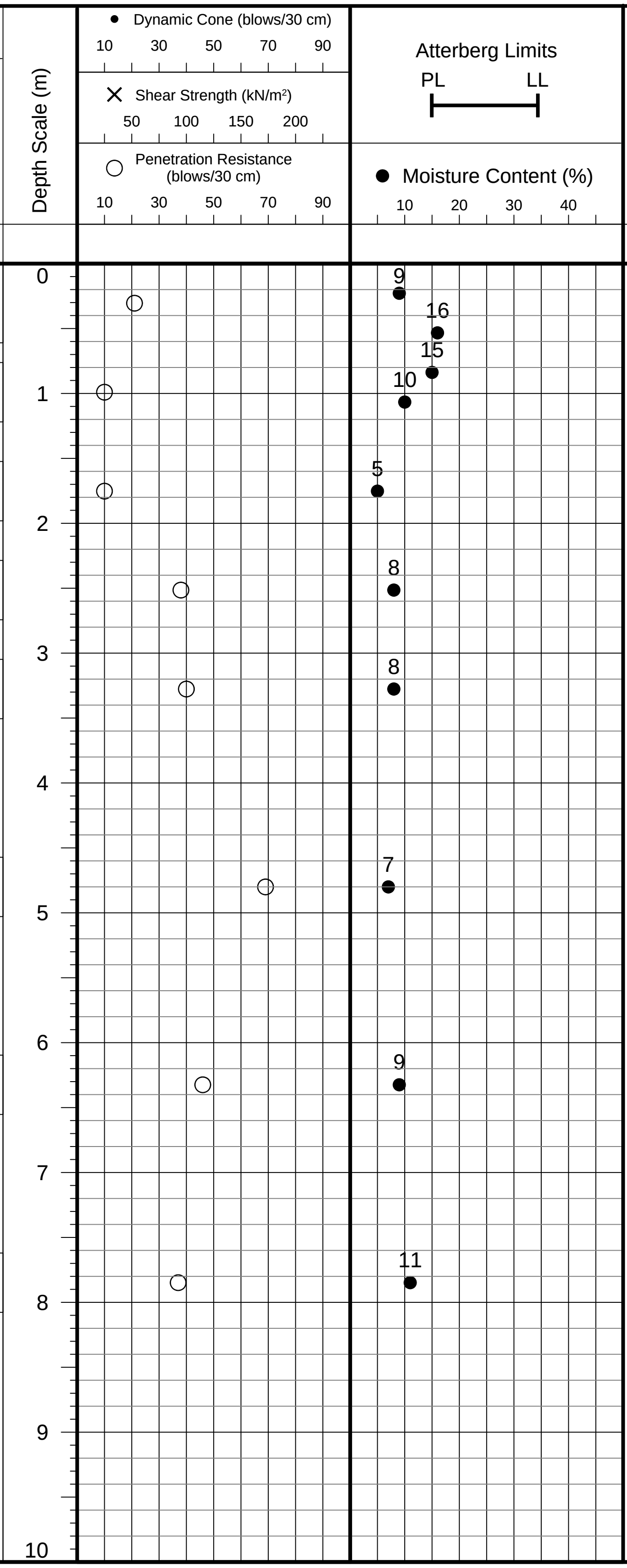
METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 405 Essa Road
City of Barrie

DRILLING DATE: July 29, 2020

El. (m) Depth (m)	SOIL DESCRIPTION
312.9	Ground Surface
0.0	Brown **EARTH FILL** **(Silty Sand)** traces of clay and gravel with organic inclusions
0.5	Dark brown **TOPSOIL**
312.0 0.9	Brown, loose, weathered **SAND** fine grained some silt
311.2 1.7	Loose to very dense **SILTY SAND TILL** traces of clay and gravel occ. sand seams and layers, cobbles and boulders — — weathered — — brown grey
305.3 7.6	Grey, dense **SILTY FINE SAND** a trace of clay with clay and silt layers
304.8 8.1	**END OF BOREHOLE**

SAMPLES		
Number	Type	N-Value
1A	DO	21
1B		
2A 2B	DO	10
3	DO	10
4	DO	38
5	DO	40
6	DO	69
7	DO	46
8	DO	37



WATER LEVEL

∇ W.L. @ El. 305.0 m on completion of the field work

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Page: 1 of 1

JOB NO.: 2007-S085

LOG OF BOREHOLE NO.: 102

FIGURE NO.: 2

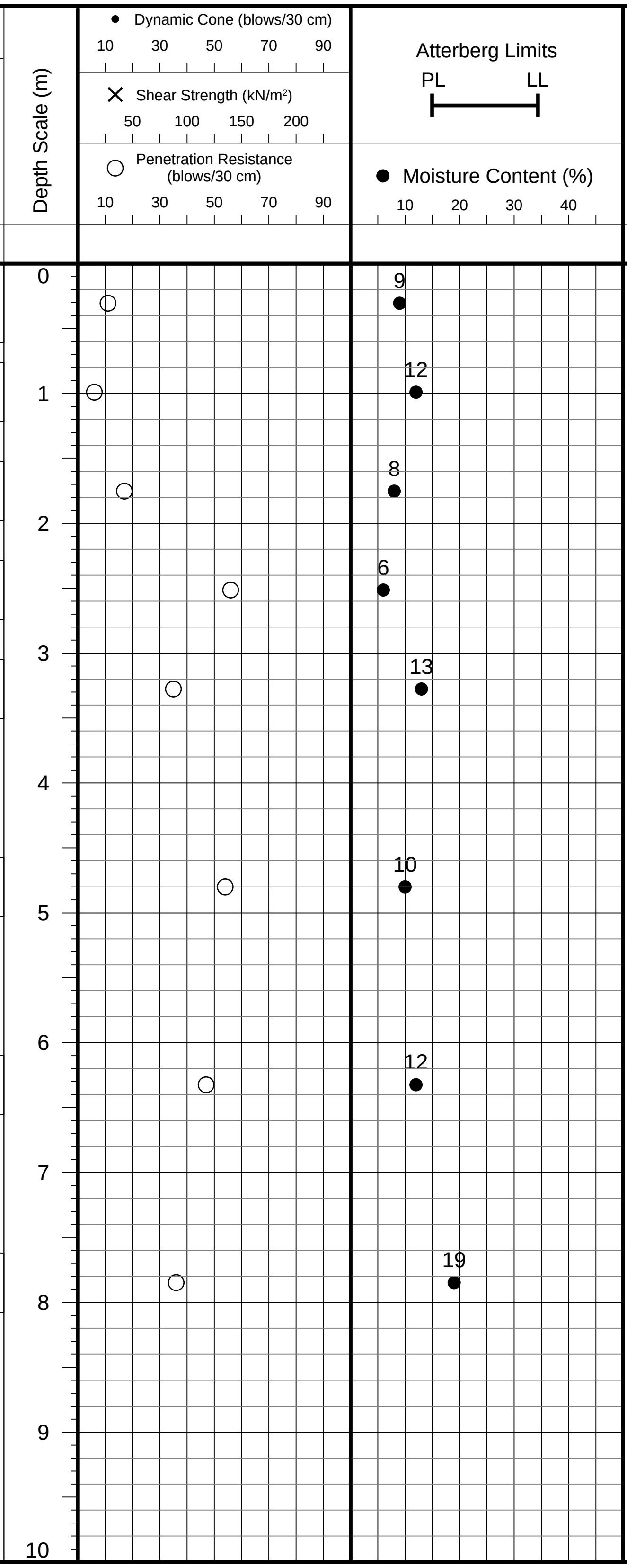
PROJECT DESCRIPTION: Proposed Residential Building
with Underground Parking

PROJECT LOCATION: 405 Essa Road
City of Barrie

METHOD OF BORING: Solid Stem Augers

DRILLING DATE: July 29, 2020

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
312.5	Ground Surface			
0.0	**15 cm TOPSOIL** Brown, compact, weathered **SAND** fine grained some silt occ. gravel with organic inclusions	1	DO	11
311.7 0.8	Brown, loose to compact **SILTY SAND TILL** traces of clay and gravel occ. sand seams and layers, cobbles and boulders	2	DO	6
	weathered	3	DO	17
310.2 2.3	Brown, very dense **SAND** fine grained a trace of silt occ. gravel	4	DO	56
309.5 3.0	Dense to very dense **SILTY FINE SAND** a trace of clay with silt layers	5	DO	35
	brown grey	6	DO	54
		7	DO	47
		8	DO	36
304.4 8.1	**END OF BOREHOLE**			



WATER LEVEL

W.L. @ El. 304.5 m on completion of the field work ▽

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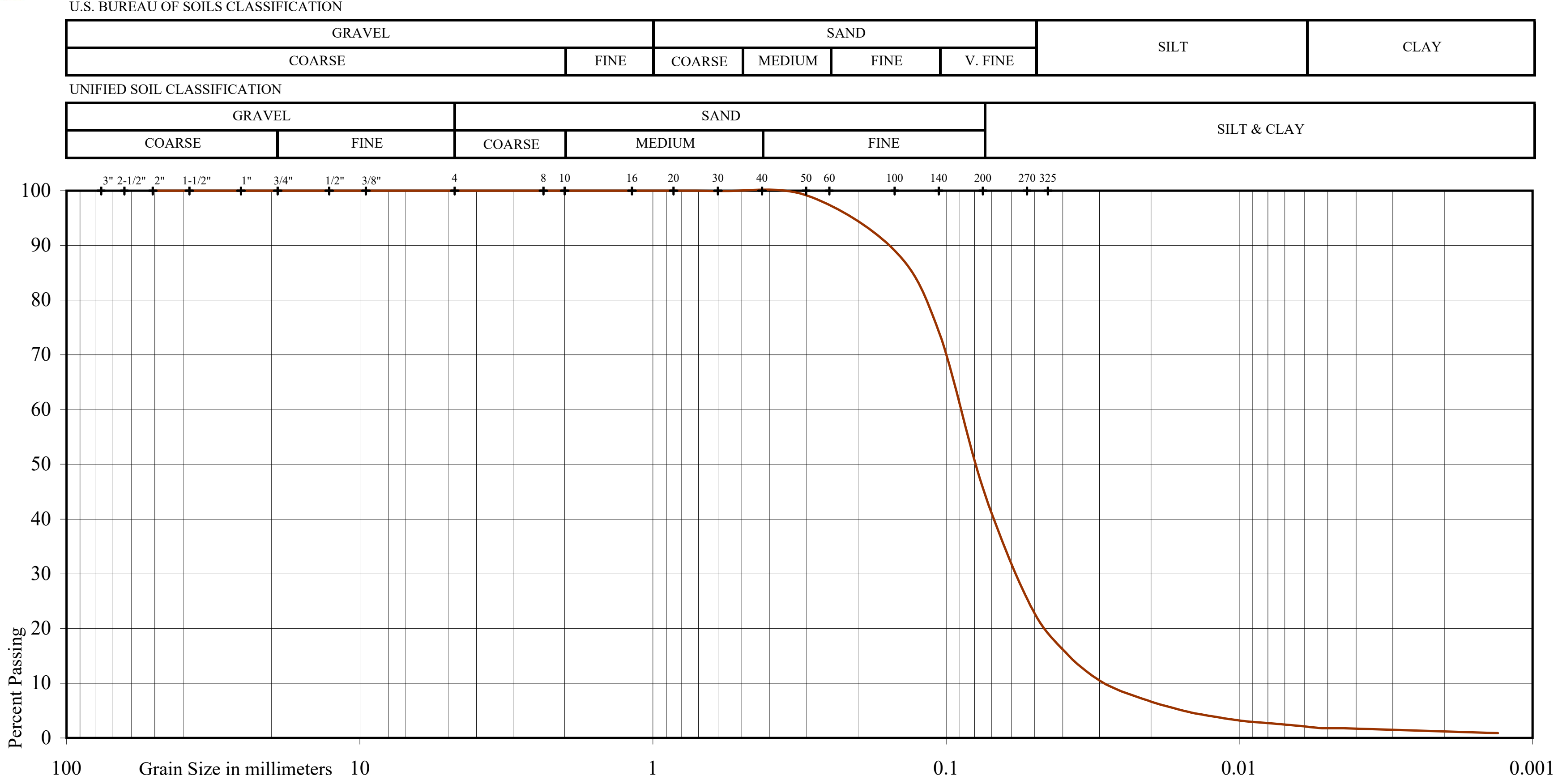
GRAIN SIZE DISTRIBUTION

Reference No: 2007-S085



Project: Proposed Residential Building with Underground Parking

Location: 405 Essa Road, City of Barrie

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Borehole No: 102

Sample No: 6

Moisture Content (%) = 10

Depth (m): 4.8

Estimated Permeability

Elevation (m): 307.7

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

Classification of Sample [& Group Symbol]: SILTY FINE SAND, a trace of clay

Figure: 3

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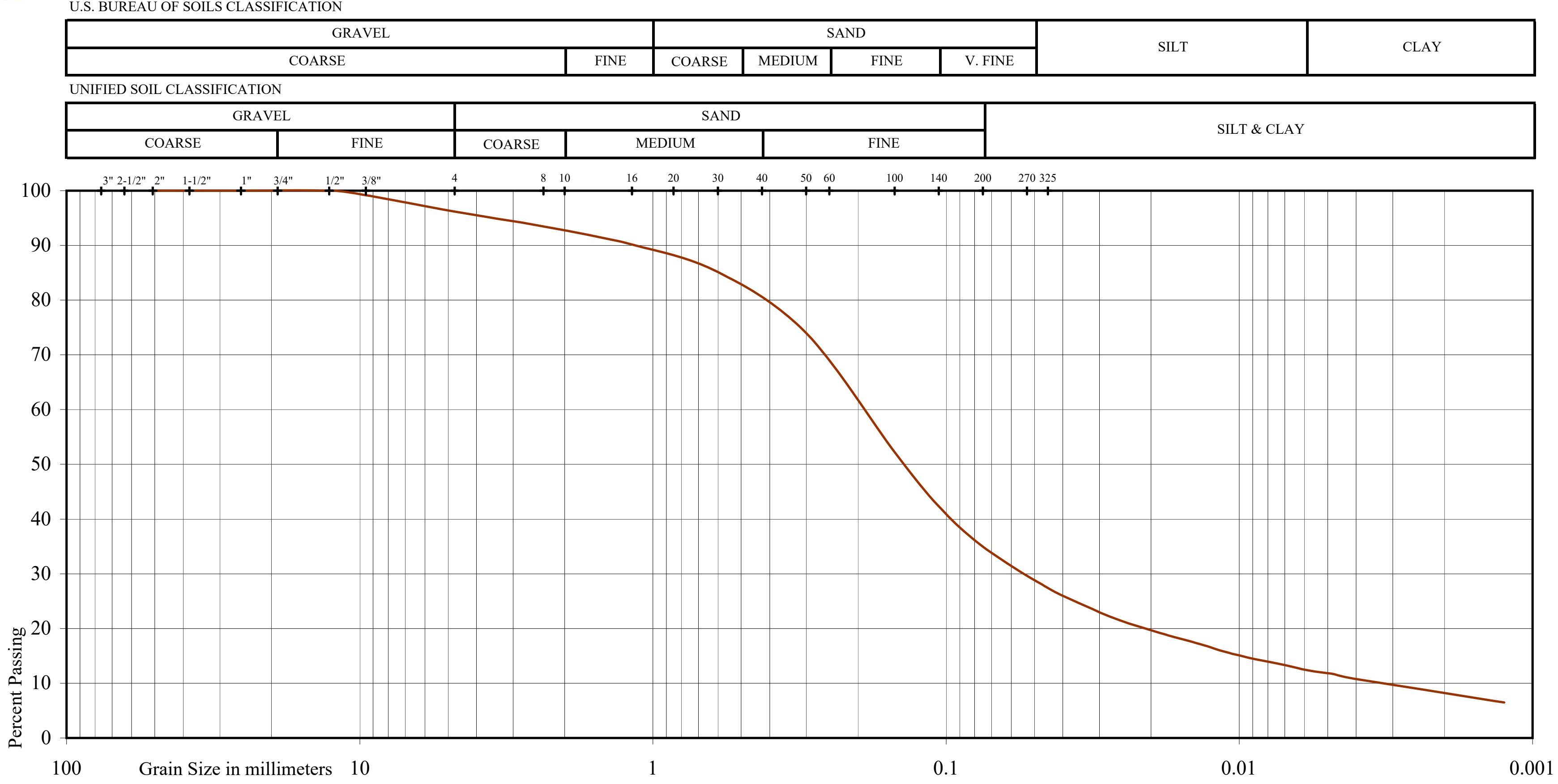
GRAIN SIZE DISTRIBUTION

Reference No: 2007-S085



Project: Proposed Residential Building with Underground Parking

Location: 405 Essa Road, City of Barrie

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 9

Estimated Permeability

(cm./sec.) = 10^{-5}

Borehole No: 101

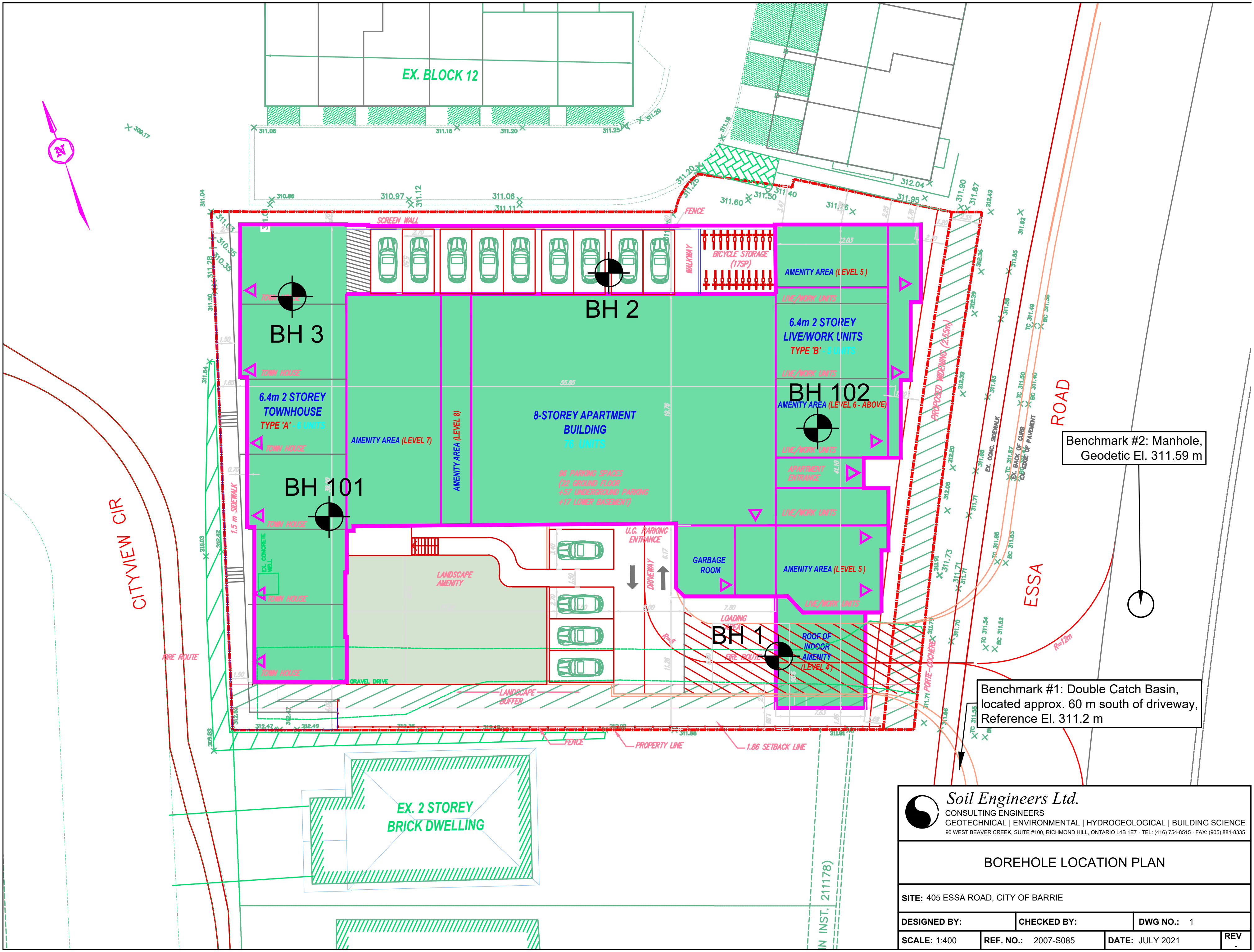
Sample No: 7

Depth (m): 6.4

Elevation (m): 306.5

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

Figure: 4



Benchmark #2: Manhole, Geodetic El. 311.59 m

Benchmark #1: Double Catch Basin, located approx. 60 m south of driveway, Reference El. 311.2 m

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BOREHOLE LOCATION PLAN

SITE: 405 ESSA ROAD, CITY OF BARRIE

DESIGNED BY:	**CHECKED BY:**	**DWG NO.:** 1	
SCALE: 1:400	**REF. NO.:** 2007-S085	**DATE:** JULY 2021	**REV** -

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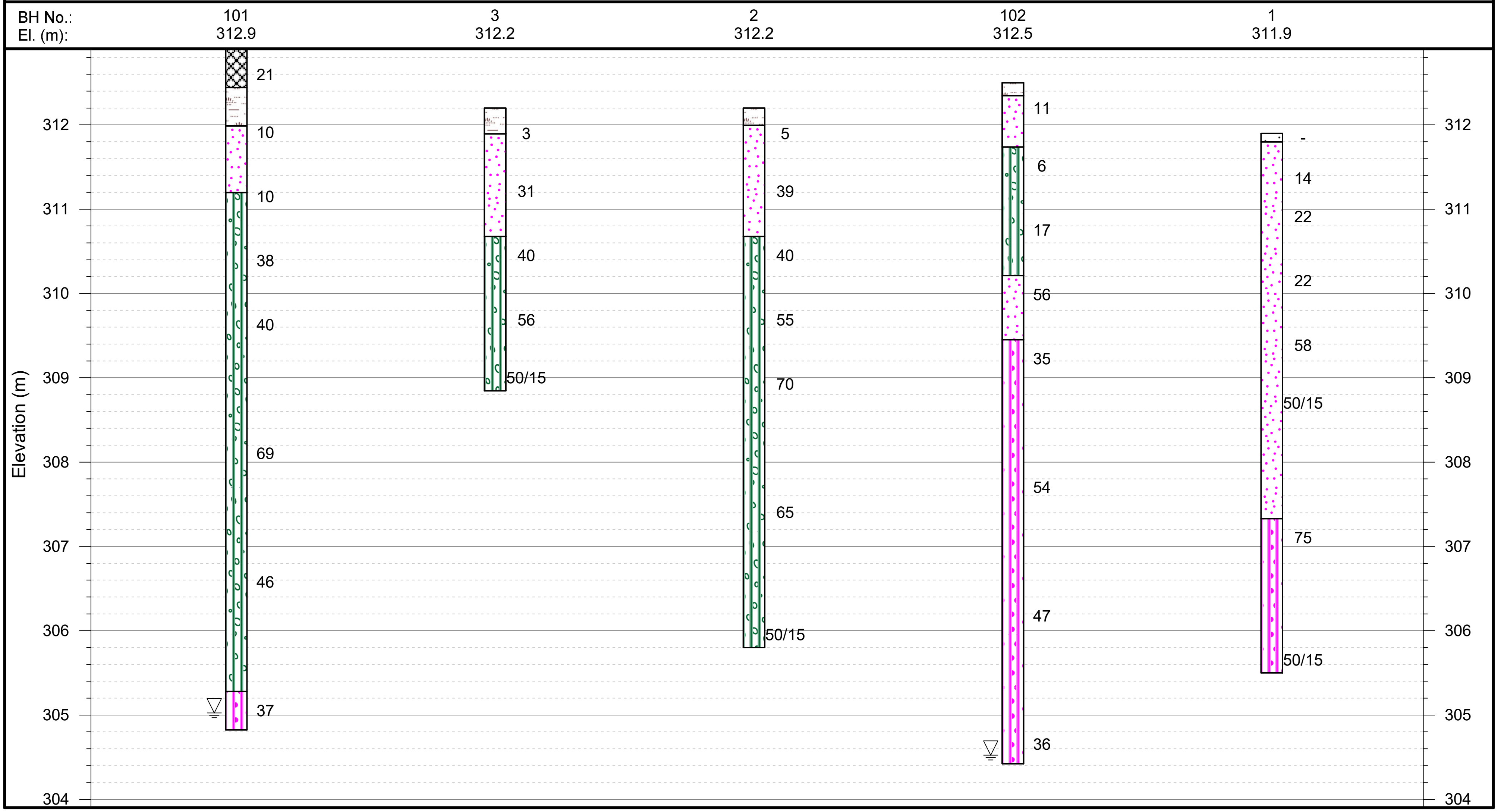
SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

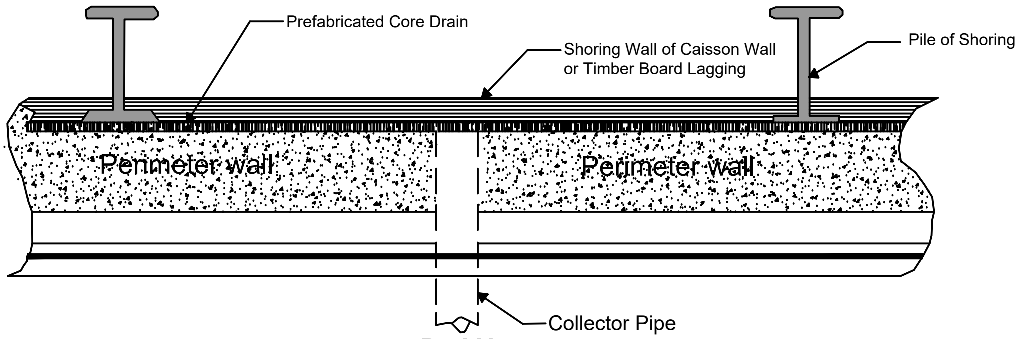
JOB NO.: 2007-S085
REPORT DATE: July 2021
PROJECT DESCRIPTION: Proposed Residential Building
with Underground Parking
PROJECT LOCATION: 405 Essa Road
City of Barrie

LEGEND

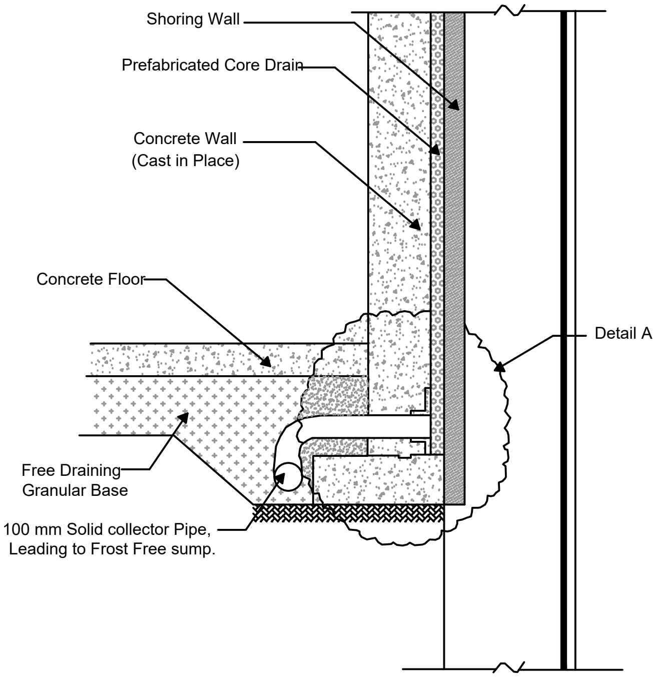
▤	TOPSOIL	▩	FILL	▥	SILTY FINE SAND	▥	SILTY SAND TILL
⁘	GRANULAR	⁘	SAND				

∇ WATER LEVEL (END OF DRILLING)

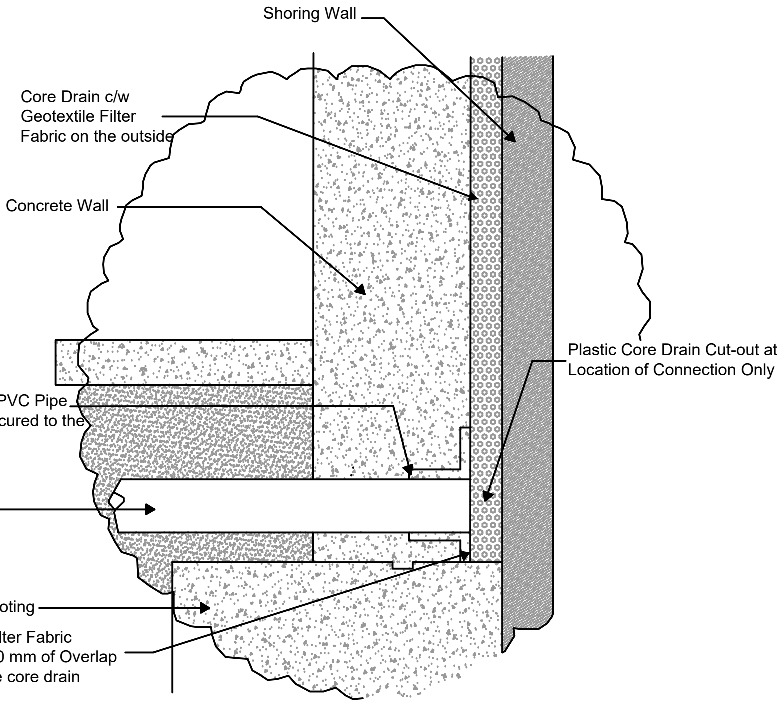




PLAN



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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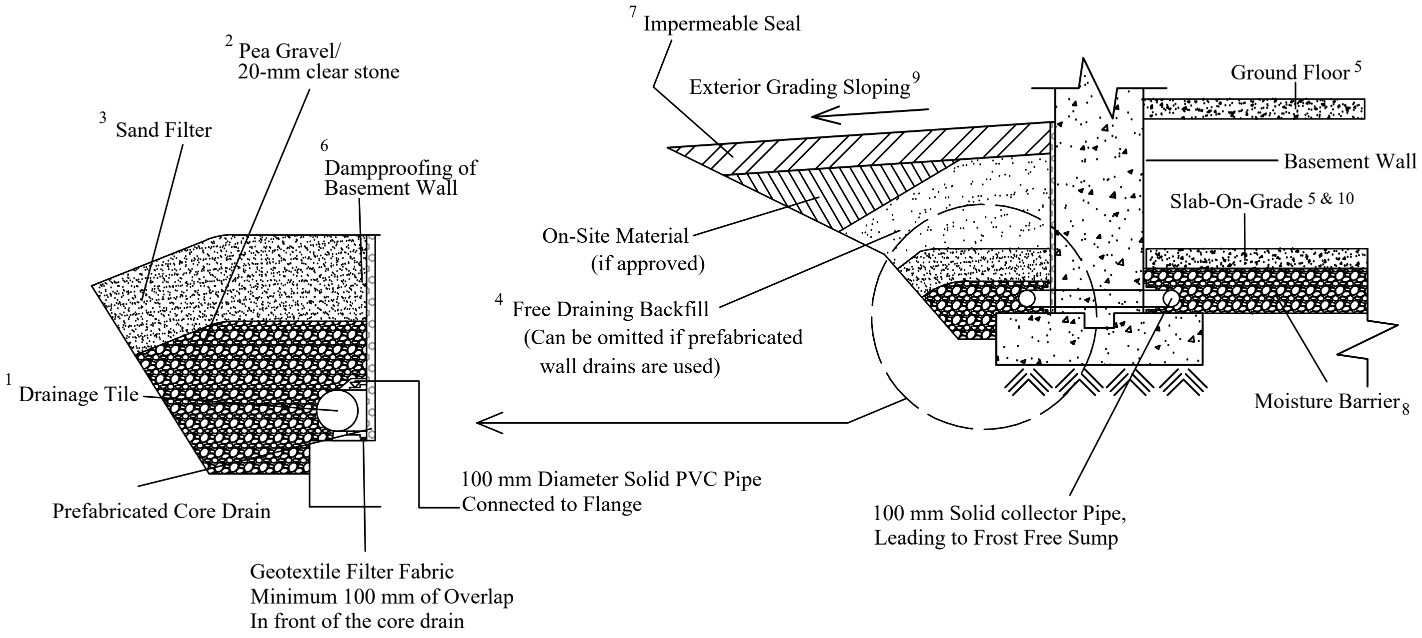
PERMANENT PERIMETER DRAINAGE SYSTEM
(WITH SHORING)

SITE: 405 ESSA ROAD, CITY OF BARRIE

DESIGNED BY: K.L. **CHECKED BY:** B.S. **DWG NO.:** 3

SCALE: N.T.S. **REF. NO.:** 2007-S085 **DATE:** JULY 2021 **REV** -



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 slab. Weep holes connections should be provided through the foundation walls for the connection between the weeping tile and the collector pipe at 20 m c/c.

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 soils, pavement or sidewalk. 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 200 mm (8") 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.

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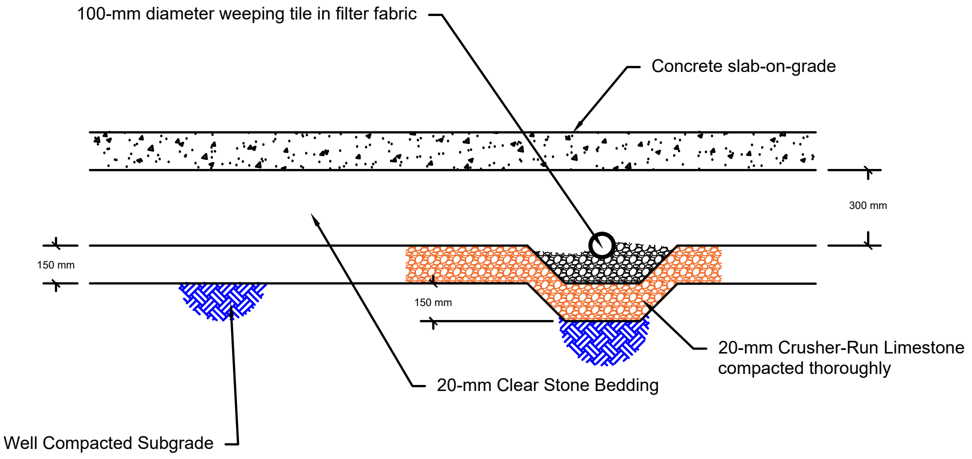
PERMANENT PERIMETER DRAINAGE SYSTEM
(WITHOUT SHORING)

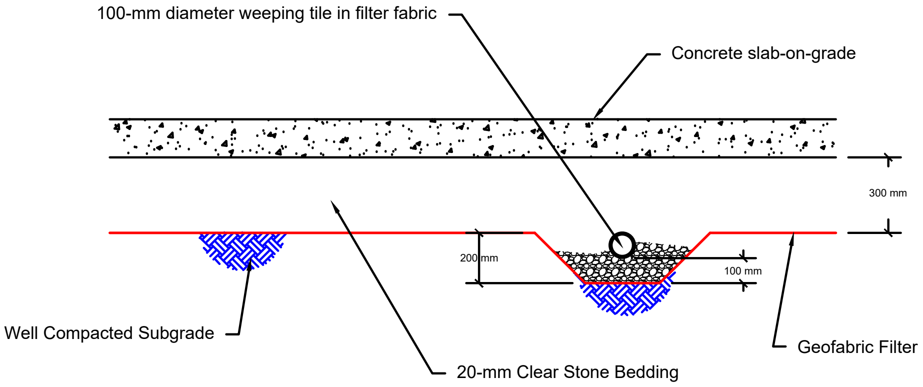
SITE: 405 ESSA ROAD, CITY OF BARRIE

DESIGNED BY: K.L. **CHECKED BY:** B.S. **DWG NO.:** 4

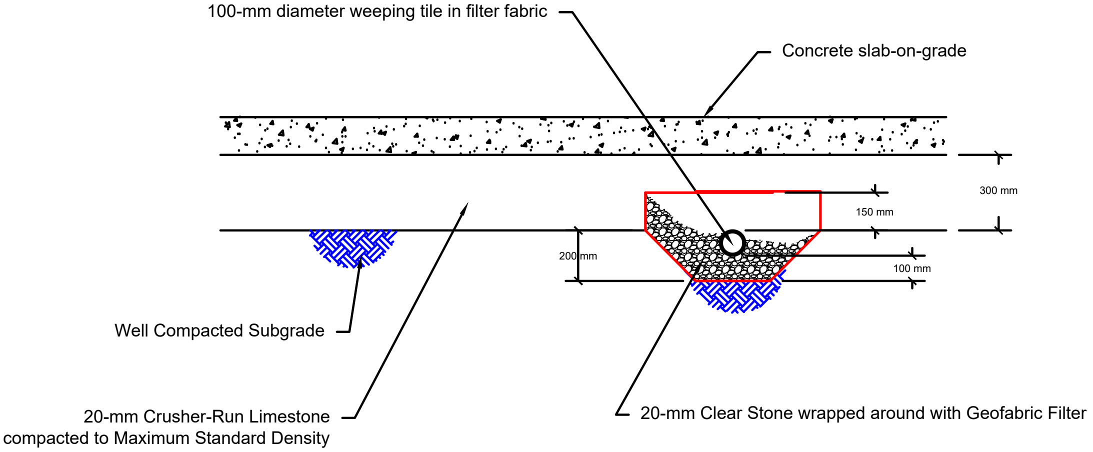
SCALE: N.T.S. **REF. NO.:** 2007-S085 **DATE:** JULY 2021 **REV** -



Option 'A'



Option 'B'



Option 'C'

Note:

1. Weepers should be placed in 6 m grids, draining in a positive gradient towards an outlet or a sump pit for removal by pumping.
2. A 10-mil polyethylene sheet should be specified between the gravel bedding and concrete slab.

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DETAILS OF UNDERFLOOR WEEPERS

SITE: 405 ESSA ROAD, CITY OF BARRIE

DESIGNED BY: K.L.
CHECKED BY: B.S.
DWG NO.: 5
SCALE: N.T.S.
REF. NO.: 2007-S085
DATE: JULY 2021
REV -

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APPENDIX ‘A’

BOREHOLE LOGS AND
GRAIN SIZE DISTRIBUTION GRAPHS FROM
GEOTECHNICAL INVESTIGATION REPORT
REFERENCE NO. 1710-S170
DATED FEBRUARY 2018

REFERENCE NO. 2007-S085

JOB NO.: 1710-S170

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

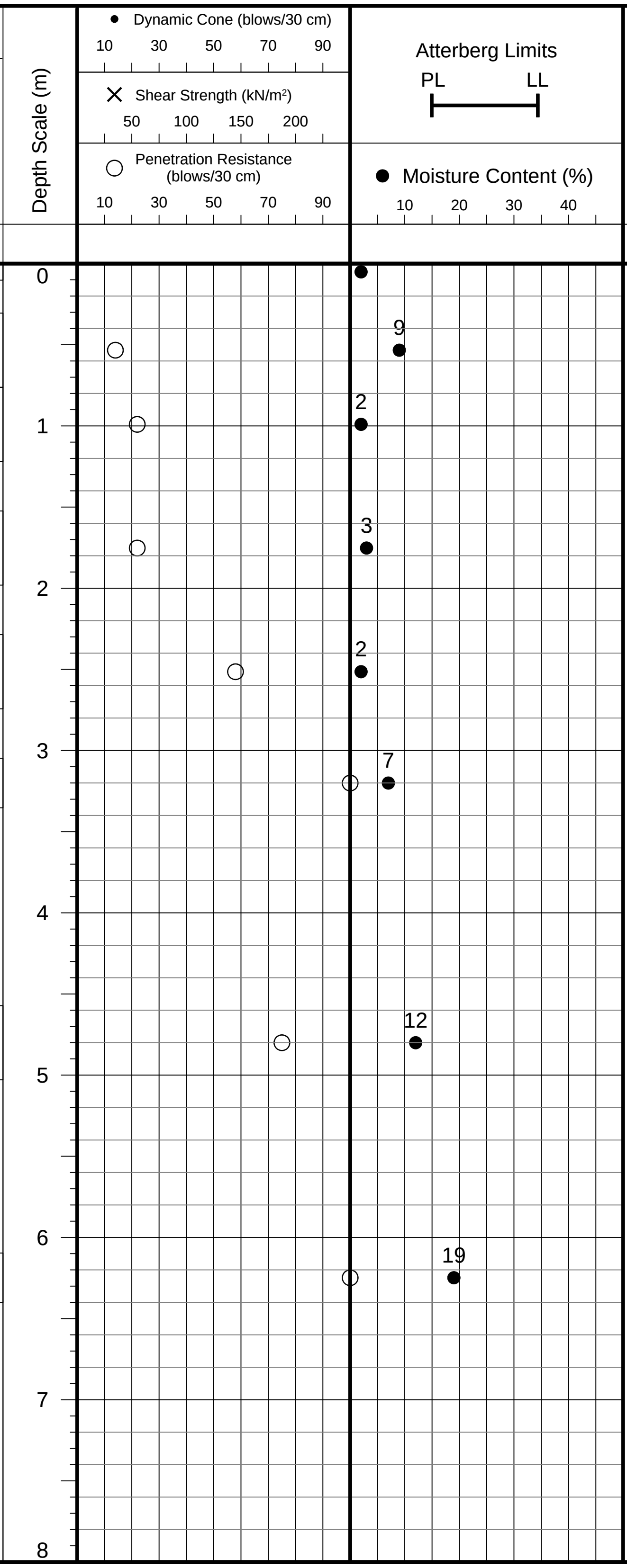
PROJECT DESCRIPTION: Proposed Residential Development - Phase 3

METHOD OF BORING: Flight-Auger
(Solid Stem)

PROJECT LOCATION: 405 Essa Road
City of Barrie

DRILLING DATE: November 20, 2017

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
311.9	Ground Surface			
0.0	**100 mm GRANULAR FILL**	1	AS	-
	Brown, compact to very dense	2	DO	14
		3	DO	22
	SAND weathered	4	DO	22
	fine grained a trace to some silt occ. gravel	5	DO	58
	silt layers	6	DO	50/15
307.3 4.6	Brown, very dense **SILTY FINE SAND** a trace of clay	7	DO	75
		8	DO	50/15
305.5 6.4	**END OF BOREHOLE**			



WATER LEVEL

Dry on completion

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Page: 1 of 1

JOB NO.: 1710-S170

LOG OF BOREHOLE NO.: 2

FIGURE NO.: 2

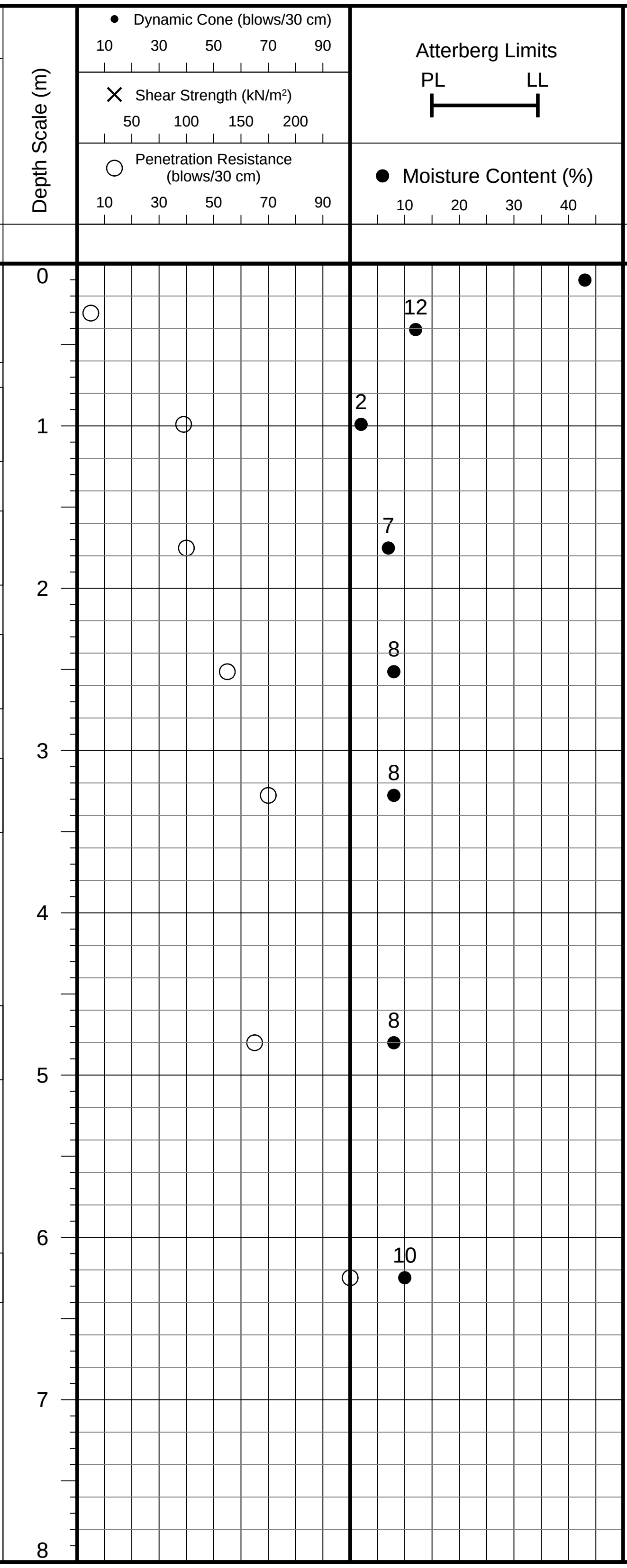
PROJECT DESCRIPTION: Proposed Residential Development - Phase 3

METHOD OF BORING: Flight-Auger (Solid Stem)

PROJECT LOCATION: 405 Essa Road
City of Barrie

DRILLING DATE: November 20, 2017

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
312.2	Ground Surface			
0.0	**20 cm TOPSOIL**	1A	DO	5
	Brown, loose to dense **SAND** weathered fine grained a trace to some silt occ. gravel	1B		
		2	DO	39
310.7 1.5	Brown, dense to very dense **SILTY SAND TILL** traces to some clay and gravel occ. wet sand and silt seams and layers, cobbles and boulders	3	DO	40
		4	DO	55
		5	DO	70
		6	DO	65
	clay layers	7	DO	50/15
305.8 6.4	**END OF BOREHOLE**			



WATER LEVEL

Dry on completion

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JOB NO.: 1710-S170

LOG OF BOREHOLE NO.: 3

FIGURE NO.: 3

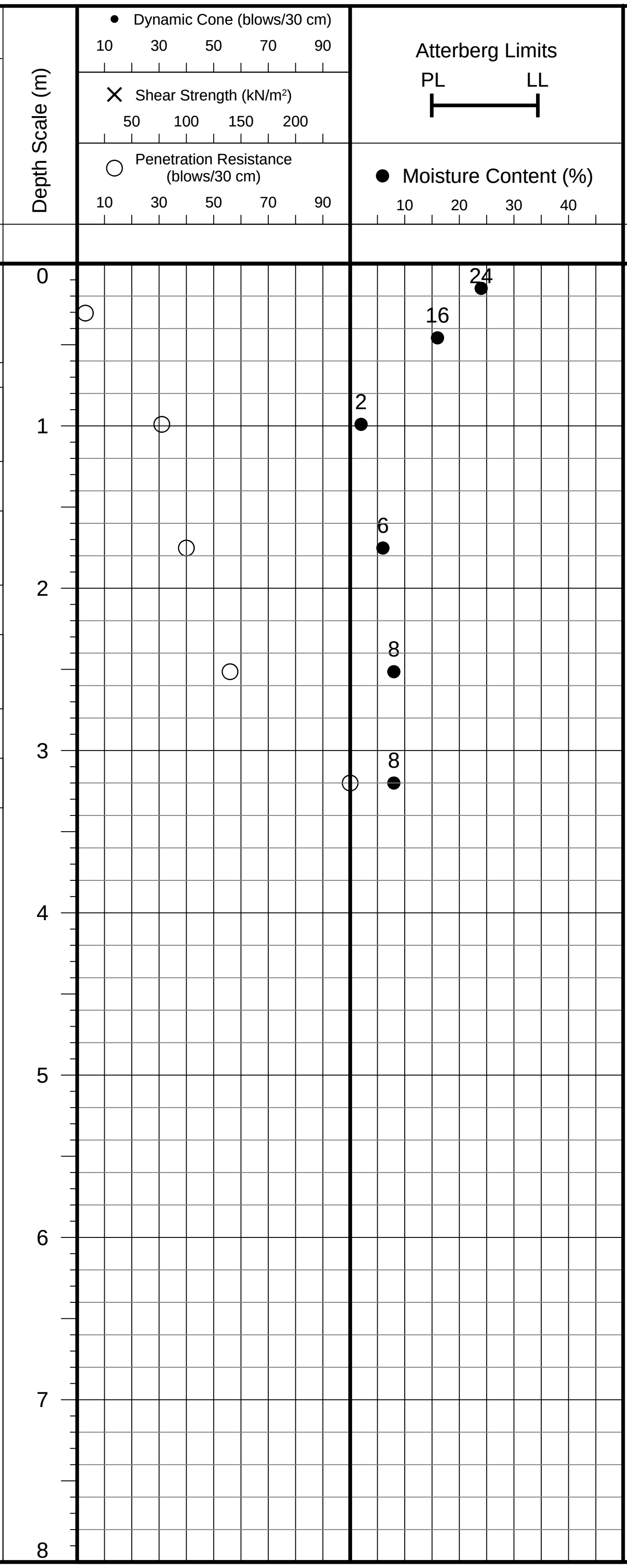
PROJECT DESCRIPTION: Proposed Residential Development - Phase 3

METHOD OF BORING: Flight-Auger
(Solid Stem)

PROJECT LOCATION: 405 Essa Road
City of Barrie

DRILLING DATE: November 20, 2017

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES			WATER LEVEL
		Number	Type	N-Value	
312.2	Ground Surface				
0.0	**30 cm TOPSOIL**	1A	DO	3	
	Brown, very loose to dense **SAND** organic inclusions/ weathered	1B			
	fine grained a trace to some silt occ. gravel	2	DO	31	
310.7 1.5	Brown, dense to very dense **SILTY SAND TILL**	3	DO	40	
		4	DO	56	Dry on completion
	traces of clay and gravel occ. wet sand and silt seams and layers, cobbles and boulders	5	DO	50/15	
308.8 3.4	**END OF BOREHOLE**				



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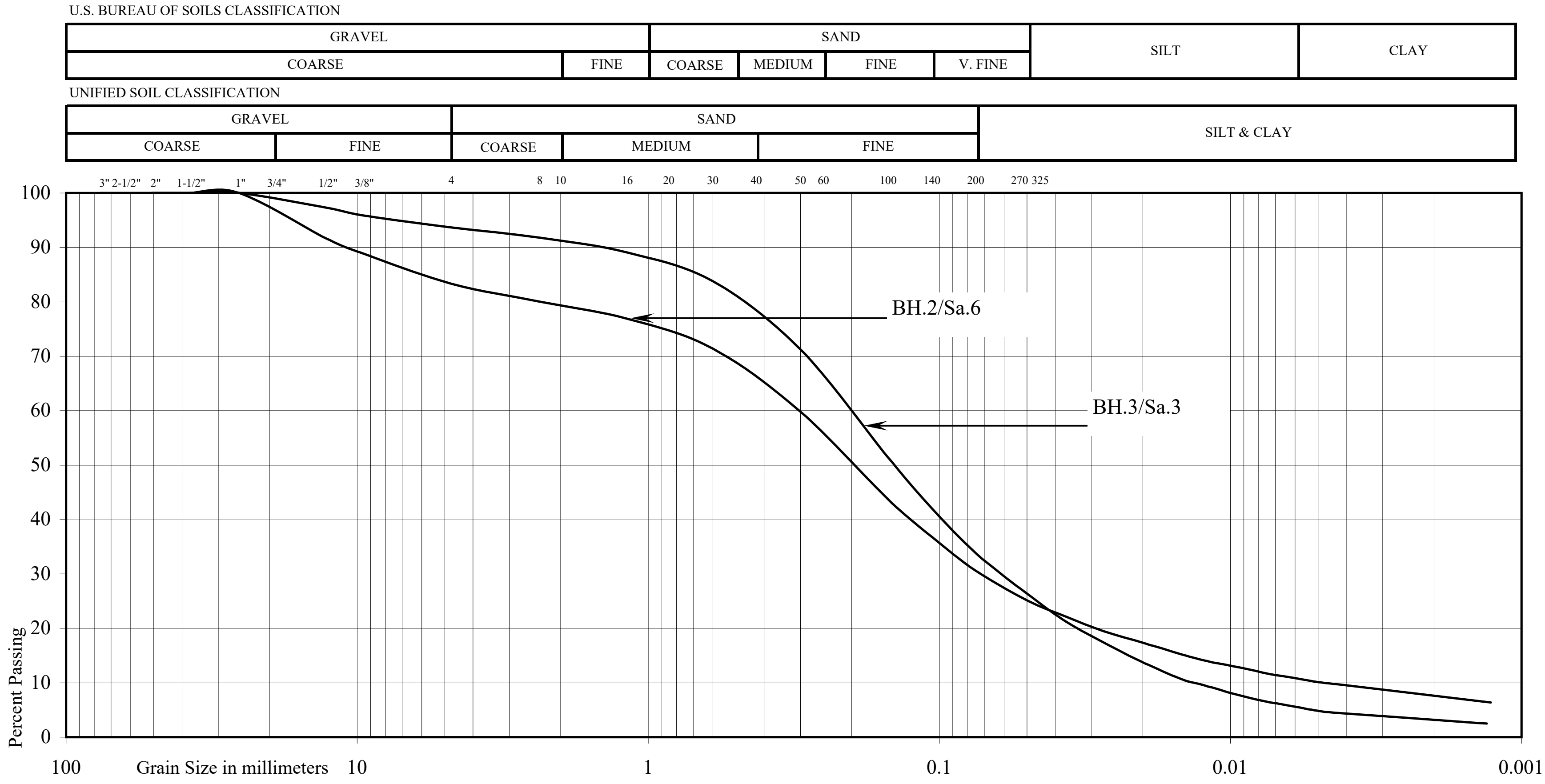
GRAIN SIZE DISTRIBUTION

Reference No: 1710-S170



Project: Proposed Residential Development - Phase 3

Location: 405 Essa Road, City of Barrie

Borehole No:	2	3
Sample No:	6	3
Depth (m):	4.8	1.8
Elevation (m):	307.4	310.4

BH./Sa.	2/6	3/3
Liquid Limit (%) =	-	-
Plastic Limit (%) =	-	-
Plasticity Index (%) =	-	-
Moisture Content (%) =	8	6
Estimated Permeability (cm./sec.) =	10^{-5}	10^{-4}

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

Figure: 4

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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) 542-2769

APPENDIX 'B'

SHORING DESIGN

REFERENCE NO. 2007-S085

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 areas in close proximity to adjacent structures and where the excavation will be extending below the foundation level so that any movement in the adjacent properties is a concern, or in an excavation embedding into saturated sand or silt deposits, 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 B1. The design soil parameters are provided in the geotechnical report.

The lateral earth pressure expressions do not include hydrostatic pressure build up behind the shoring. If the wall is designed to be water tight 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 expression:

In Cohesionless Soils: $R = 1.5\ D\ K_p\ L^2\ \gamma$

where R = Ultimate Load to be restrained (kN)
D = Diameter of concrete filled hole (m)
K_p = Passive resistance in the silt till and sand deposits
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 extending into the till deposit can be estimated using an adhesion values of 75 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 anchors.

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 till deposit below the bottom of excavation should be designed for the allowable bearing pressure of 250 kPa.

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.

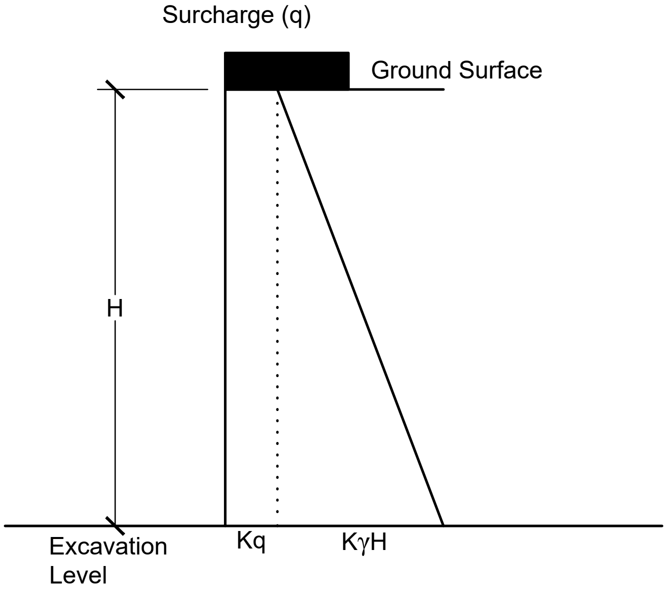
To prevent undermining of the raker footing, no excavation should be made within two times the width of raker footing on the opposite side of the raker.

When sloping berm excavation procedures are used, the rakers should be installed in trenches in the berm to minimize movement of the shoring wall being supported. In addition, the rakers can be pre-loaded and secured in place before removal of the earth berm.

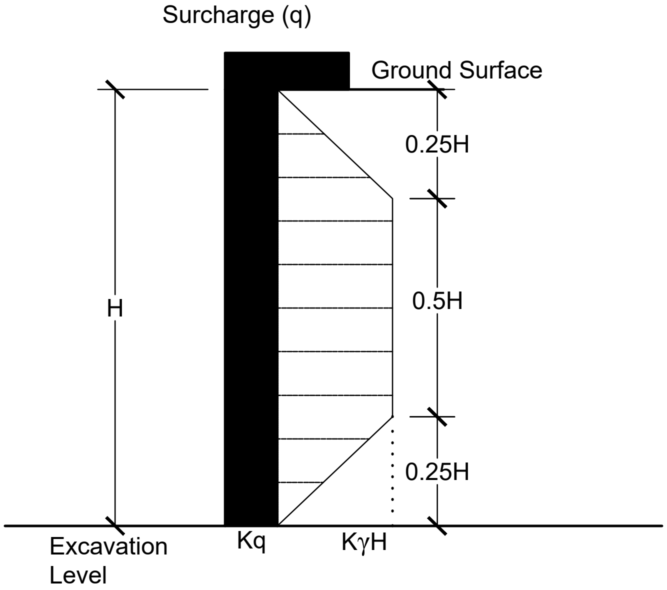
MONITORING OF PERFORMANCE

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.

TEMPORARY SHORING
Lateral Earth Pressures



Single Support System



Multiple Support System

Lateral Pressure $P = K (\gamma H + q)$

Where

H = Height of Shoring	m
γ = Unit Weight of Retained Soil	kN/m^3
q = Surcharge	kPa
K = Earth Pressure Coefficient	

- If moderate ground and shoring movements are permissible then:
  $K = K_a$ = Active Earth Pressure Coefficient

- if there are building foundations within a distance of 0.5 H behind the shoring then:
  $K = K_o$ = Earth Pressure at rest

- If there are building foundations within a distance of between 0.5 H and H behind the shoring then:
  $K = 0.5 (K_a + K_o)$

Note:

1. The lateral pressure expression assumes effective drainage from behind the temporary shoring.
2. The earth pressure coefficients are specified in the geotechnical report.

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TEMPORARY SHORING

SITE: 405 ESSA ROAD, CITY OF BARRIE

DESIGNED BY: K.L. **CHECKED BY:** B.S. **DWG NO.:** B1

SCALE: N.T.S. **REF. NO.:** 2007-S085 **DATE:** JULY 2021 **REV** -