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A REPORT TO
2596843 ONTARIO INC.

A GEOTECHNICAL INVESTIGATION FOR PROPOSED MIXED-USE DEVELOPMENT

224 ARDAGH ROAD

CITY OF BARRIE

Reference No. 1802-S072

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1.0 INTRODUCTION

In accordance with the written authorization dated February 22, 2018, from Mr. John Stante, President of 2596843 Ontario Inc., a geotechnical investigation was carried out on a parcel of land located at 224 Ardagh Road in the City of Barrie.

The purpose of the investigation was to reveal the subsurface conditions and to determine the engineering properties of the disclosed soils for the design and construction of mixed use development. The findings and resulting geotechnical recommendations for a 3-storey mixed use building with 1-level underground parking have been presented in the Report dated April 2018.

The latest design of the development will consist of a 6-storey mixed use building and 31 freehold townhouse units, with both at grade and underground parking in the mixed use building. The recommendations for the design and construction of the development are revised 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 lacustrine silts and clay.

The subject property is situated at the northwest intersection of Ardagh Road and Ferndale Drive South in the City of Barrie. It is currently a vacant field with weed cover. An earth berm of almost 3 m in height was evident at the south portion of the property and a paved parking lot was present at the east portion at the time of investigation.

We understand that the proposed development will consist of a 6-storey mixed use building and 31 freehold townhouse units, with both at grade and underground parking in the mixed use building. The development will be provided with municipal services and access driveway.

3.0 FIELD WORK

The field work, consisting of five (5) sampled boreholes, was performed on March 27 and 28, 2018, at the locations shown on the Borehole Location Plan, Drawing No. 1. These boreholes extended to a depth of 6.5 m or 8.1 m, having two of the boreholes extended deeper to 9.6 m and 10.8 m.

The boreholes were advanced at intervals to the sampling depths by a track-mounted, continuous-flight power-auger machine with hollow stem augers 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.

Upon completion of drilling and sampling, four (4) monitoring wells were installed in selected boreholes for hydrogeological assessment. Parts of the well record are included in Section 5.0. The findings are included in the Hydrogeological report under a separate cover.

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

The ground elevation at each borehole location was interpolated from the Topography Plan prepared by Guido Papa Surveying dated February 28, 2018.

4.0 SUBSURFACE CONDITIONS

Detailed descriptions of the encountered subsurface conditions are presented on the Borehole Logs, comprising Figures 1 to 5, inclusive. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing No. 2, and the engineering properties of the disclosed soils are discussed herein.

The investigation has disclosed that beneath the surface course of topsoil or asphalt pavement, a layer of earth fill was contacted in places and the site was underlain by deposits of sand and silt with occasional layers of silty clay.

4.1 **Topsoil** (Boreholes 1, 3, 4, and 5)

The revealed topsoil is 160 to 210 mm thick at the borehole locations. The existing earth berm at the south portion of the site could also contain topsoil, which have to be reviewed from test pit excavation.

The topsoil is dark brown in colour, indicating appreciable amounts of roots and humus. These materials are unstable and compressible under loads; therefore, the topsoil is considered to be void of engineering value. It can only be used for general landscaping and landscape contouring purposes.

Due to its humus content, the topsoil may produce volatile gases and generate an offensive odour under anaerobic conditions. Therefore, the topsoil must not be buried below any structure, or deeper than 1.2 m below the finished grade, so that it will not have an adverse impact on the environmental well-being of the developed areas.

4.2 **Pavement Structure** (Borehole 2)

The existing pavement of the parking lot at the borehole location consists of asphalt of approximately 80 mm thick and a granular base of 530 mm thick.

4.3 **Earth Fill** (Boreholes 1, 2 and 5)

The revealed earth fill extends to a depth of 0.6 and 1.0± m at the locations of Boreholes 1 and 5. At Borehole 2 location, the earth fill extends to a depth of 4.9 m from the prevailing ground surface.

The fill consists of sand or silty sand with gravel. Concrete rubble was also contacted in the earth fill at the location of Borehole 2, which may represent the backfill of a cavity, probably a previous house foundation at this location.

The obtained 'N' values range from 3 to 16, with a median of 10 blows per 30 cm of penetration, indicating the fill was non-uniform in compaction. The natural water content of the earth fill samples was determined; the results are plotted on the Borehole Logs, ranging from 6% to 25%, indicating moist to wet conditions.

The fill is amorphous in structure; it will ravel and is susceptible to collapse in steep cuts. For structural uses, the earth fill must be subexcavated, sorted free of topsoil and any deleterious material and properly compacted in layers.

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 encountered below the topsoil or earth fill at the locations of Boreholes 1, 3, 4 and 5, at a depth of 0.2 to 1.0 m from grade. It extended to a depth of 2.5 m to 4.8 m from the prevailing ground surface. A lower sand deposit was contacted at locations of Boreholes 1, 2, 4 and 5, below the silt deposit at 4.4 to 9.5 m, extending to the termination depth of the boreholes.

The upper sand deposit is generally fine to medium grained and the lower sand deposit is fine grained, with variable amount of silt.

The Standard Penetration 'N' values range from 7 to more than 100, with a median of 27 blows per 30 cm of penetration, indicating it is loose to very dense, being generally compact in relative density.

The natural water content values of the sand samples were found to range from 4% to 24%, with a median of 9%, showing the sand deposit is damp to moist, with wet sand in localized areas and depths.

Grain size analysis was performed on 1 representative sample; the gradation result is plotted on Figure 6.

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

- Low to high frost susceptibility, depending on the silt content.
- Pervious, with an estimated coefficient of permeability of 10^{-2} to 10^{-3} cm/sec, a percolation rate of 5 to 10 min/cm and runoff coefficients of:

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

- A frictional soil, its shear strength is derived from its internal friction angle and is soil density dependent.
- In excavation, the sand will slough to its angle of repose, run under seepage pressure and boil with a piezometric head of about 0.3 m.
- A good pavement-supportive material, with an estimated CBR value of 16%.
- Low corrosivity to buried metal, with an estimated electrical resistivity of 6000 ohm·cm.

4.5 **Silt** (Boreholes 1, 2, 4 and 5)

The silt deposit was contacted in the sand stratum or below the earth fill at 2.5 m to 4.9 m from grade. It is fine grained, with fine sand seams and occasional clay layers.

The obtained ‘N’ values range from 16 to 63, with a median of 19 blows per 30 cm of penetration, indicating that the relative density of the silt is compact to very dense, being generally compact.

The natural water content values of the silt samples range from 9% to 25%, with a median of 19%, indicating the silt is very moist to wet, which becomes dilatant when shaken by hand.

Grain size analyses were performed on 2 representative samples, the gradation results are plotted on Figure 7.

Based on the above findings, the engineering properties of the silt deposit are given below:

- High capillarity and water retention capability.
- High frost susceptibility and soil-adfreezing potential.
- High water erodibility; it will migrate through small openings under moderate seepage pressure.
- Relatively low permeable, with an estimated coefficient of permeability of 10^{-3} to 10^{-4} cm/sec, a percolation rate of 10 to 12 min/cm 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

- A frictional soil, its shear strength is density dependent. Due to its dilatancy, the strength of the wet silt 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 in shear strength.
- In excavation, the silt will slough and run slowly with seepage bleeding from the cut face. It will boil with a piezometric head of 0.4 m.
- A poor pavement-supportive material, with an estimated CBR value of 3%.
- Moderate to moderately low corrosivity to buried metal, with an estimated electrical resistivity of 4000 to 5500 ohm·cm.

4.6 **Silty Clay** (Borehole 3)

The silty clay was contacted below the sand deposit at a depth of 4.8 m from the existing ground surface. It is cohesive, with seams of fine sand and silt. The

Standard Penetration ‘N’ values are 12 and 15 blows per 30 cm of penetration, showing a stiff consistency.

The natural water content of the clay samples was determined at 22% and 25%, indicating very moist conditions.

A grain size analysis was performed on a representative sample; the result is plotted on Figure 8.

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

- Highly frost susceptible and high soil-adfreezing potential.
- Virtually impervious, with an estimated coefficient of permeability less than 10^{-7} cm/sec, an average percolation rate of more than 80 min/cm, and runoff coefficients of:

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

- A cohesive-frictional soil, its shear strength is derived from consistency and augmented by the internal friction of the silt. Its shear strength is moisture dependent and, due to the dilatancy of the silt, the overall shear strength of the silty clay is susceptible to impact disturbance, i.e., the disturbance will induce a build-up of pore pressure within the soil mantle, resulting in soil dilation and a reduction of shear strength.
- In excavation, the stiff clay is relatively stable in steep cut. However, long exposure will allow the sand or silt seams to become saturated which may lead to localized sloughing.

- A poor supportive-pavement material, with an estimated CBR value of 5%.
- Moderately high corrosivity to buried metal, with an estimated electrical resistivity of 3000 ohm·cm.

4.7 Compaction Characteristics of the Revealed Soils

The obtainable degree of compaction is primarily dependent on the soil moisture and, to a lesser extent, on the type of compactor used and the effort applied. As a general guide, the typical water content values of the revealed soils for Standard Proctor compaction are presented in Table 1.

Table 1 - Estimated Water Content for Compaction

Soil Type	**Determined Natural Water Content (%)**	**Water Content (%) for Standard Proctor Compaction**	
		100% (Optimum)	**Range for 95% or +**
Earth Fill (Silty Sand)	6 to 25 (median 11)	11	7 to 14
Sand	4 to 24 (median 9)	10	6 to 13
Silt	9 to 25 (median 19)	13	8 to 16
Silty Clay	22 and 25	18	14 to 22

Based on the above findings, most of the on-site soils are suitable for 95% or + Standard Proctor compaction. However, the silt and clay may be too wet which will require aeration in dry and warm weather prior to reuse for structural compaction. The existing earth fill must be sorted free of deleterious materials, if encountered, prior to its use for structural backfill.

The silt and clay can be compacted by a kneading-type roller. The sand should be compacted by a smooth roller with or without vibration, depending on the water content of the soils being compacted. The lifts for compaction should be limited to 20 cm, or to a suitable thickness as assessed by test strips performed by the equipment which will be used at the time of construction.

One should be aware that with considerable effort, a 90%± Standard Proctor compaction of the wet sand and silt is achievable. Further densification is prevented by the pore pressure induced by the compactive effort; however, large random voids will have been expelled, and with time the pore pressure will dissipate and the percentage of compaction will increase. There are many cases on record where after a few months of rest, the density of the compacted mantle has increased to over 95% of its maximum Standard Proctor dry density.

If the compaction of the soils is carried out with the water content within the range for 95% Standard Proctor dry density but on the wet side of the optimum, the surface of the compacted soil mantle will roll under the dynamic compactive load. This is unsuitable for road construction since each component of the pavement structure is to be placed under dynamic conditions which will induce the rolling action of the subgrade surface and cause structural failure of the new pavement. The slab-on-grade, foundations or bedding of the underground services will be placed on a subgrade which will not be subjected to impact loads. Therefore, the structurally compacted soil mantle with the water content on the wet side or dry side of the optimum will provide adequate subgrade strength for the project construction.

5.0 GROUNDWATER CONDITIONS

The boreholes were checked for the presence of groundwater and cave-in upon their completion. Boreholes 4 and 5 were dry and open upon completion. However, wet cave-in was recorded in the remaining boreholes upon completion.

The groundwater levels in the monitoring wells were recorded on April 11, 27 and May 7, 2018. These records are presented in Table 2.

Table 2 - Groundwater Records

Borehole/ Monitoring Well No.	**Ground Elevation (m)**	**Groundwater/ Cave-in Elevation On Completion (m)**	**Groundwater Elevation in Monitoring Wells (m)**		
			April 11, 2018	**April 27, 2018**	**May 7, 2018**
1	259.8	253.7*	No Well		
2	259.9	255.9*	255.75	256.16	256.35
3	260.5	258.2*	256.17	256.33	256.45
4	261.2	Dry	<255.1**	<255.1**	<255.1**
5	261.8	Dry	<255.7**	<255.7**	255.95

* Cave-in level
** Below bottom of well

Wet cave-in occurred in Boreholes 1, 2 and 3, due to the localized wet soils. They do not represent the continuous groundwater in the vicinity.

Groundwater recorded in the monitoring wells at Boreholes 2 and 3 ranged from 255.75 to 256.45 m, or slightly more than 4 m below the prevailing ground surface. It represented perched water in the upper sand stratum. In excavation, the yield of groundwater is anticipated to be localized and in limited quantities.

6.0 **DISCUSSION AND RECOMMENDATIONS**

The investigation has disclosed that beneath the surface course of topsoil or asphalt pavement, with a layer of earth fill in places, the site was underlain by deposits of loose to very dense, generally compact sand and compact to very dense, being generally compact silt, with occasional layers of stiff silty clay.

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The proposed development will consist of a 6-storey mixed use building and 31 freehold townhouse units, with both at grade and underground parking in the mixed use building. The finished grade of the development, however, is not finalized. The geotechnical findings warranting special consideration for the proposed building are presented below:

1. Topsoil must be completely removed for the development.
2. Excavation for the townhouse foundation will extend to a depth below 1.5 to 2.0 m; the subsoil is anticipated to consist of native sand or silt with localized earth fill. The earth fill should be removed and the construction of conventional footings should extend onto the native subsoil. Alternatively, the earth fill can be upgraded into an engineered fill for footing construction after it is subexcavated, segregated free of topsoil or deleterious material and recompacted to the specified density in layers.
3. Excavation for the mid-rise building will extend to a depth below 3.5 m from grade and the subsoil is anticipated to consist of native sand or silt, capable of

supporting the 6-storey building on conventional footings or raft foundation.

4. The construction of a conventional underground parking structure will consist of subsurface drainage collecting the groundwater and dissipate into the municipal sewer system. If the municipality does not accept the discharge of groundwater into the sewer system, the underground parking should be designed with a storage cistern to collect and store up the subsurface water for irrigation or surface cleaning during the dry season. Alternatively, a submerged “tank” structure with a raft foundation to resist the hydrostatic pressure can be designed for the underground parking if on site storage cistern is not practical.
5. All excavation must be sloped back for safety. Temporary shoring will be required in excavation where a safe backing slope cannot be provided due to space limitation.

The recommendations appropriate for the project described in Section 2.0 are presented herein. One must be aware that the subsurface conditions may vary between boreholes. Should this become apparent during construction, a geotechnical engineer must be consulted to determine whether the following recommendations require revision.

6.1 **Site Preparation**

The topsoil must be completely removed. The existing earth berms should also be reviewed for topsoil composition. The reuse of topsoil will be limited to landscape areas only. Any surplus must be removed off-site.

If the site will have to be regraded or additional earth fill is required for site grading, it is generally more economical to place an engineered fill for conventional footings, sewer, pavement and slab construction. The engineering requirements for a certifiable fill are presented below:

1. All the existing topsoil and earth fill must be removed, and the subgrade must be inspected and proof-rolled prior to any fill placement. Badly weathered soils should also be subexcavated, sorted free of topsoil inclusions and any deleterious materials, if any, aerated and properly compacted in layers.
2. Inorganic soils must be used for the fill, and they must be uniformly compacted in lifts 20 cm thick to 98% or + of their maximum Standard Proctor dry density up to the proposed finished grade. The soil moisture must be properly controlled on the wet side of the optimum. If the foundations are to be built soon after the fill placement, the densification process for the engineered fill must be increased to 100% of the maximum Standard Proctor compaction.
3. If imported fill is to be used, it should comprise of inorganic soils, free of deleterious or any material with environmental issue (contamination). Any potential imported earth fill from off-site must be reviewed for geotechnical and environmental quality by the appropriate personnel as authorized by the developer or agency, before hauling to the site.
4. If the engineered fill is to be left over the winter months, adequate earth cover or equivalent must be provided for protection against frost action.
5. The engineered fill must extend over the entire graded area, and the engineered fill envelope must be clearly and accurately defined in the field and precisely documented by qualified surveyors.
6. Building foundations partially on engineered fill must be reinforced and designed by a structural engineer to properly distribute the stress induced by the abrupt differential settlement (estimated to be 15± mm) between the natural soils and engineered fill.
7. The engineered fill must not be placed during the period from late November

to early April when freezing ambient temperatures occur either persistently or intermittently. This is to ensure that the fill is free of frozen soils, ice and snow.

8. Where the fill is to be placed on a bank steeper than 1 vertical:3 horizontal, the face of the bank must be flattened to 3 + so that it is suitable for safe operation of the compactor and the required compaction can be obtained.
9. Where the ground is wet due to subsurface water seepage, an appropriate subdrain scheme must be implemented prior to the fill placement, particularly if it is to be carried out on sloping ground.
10. The fill operation must be fully supervised and monitored by a technician under the direction of a geotechnical engineer.
11. The footing and underground services subgrade must be inspected by the geotechnical consulting firm that supervised the engineered fill placement. This is to ensure that the foundations are placed within the engineered fill envelope, and the integrity of the fill has not been compromised by interim construction, environmental degradation and/or disturbance by the footing excavation.
12. Any excavation carried out in certified engineered fill must be reported to the geotechnical consultant who supervised the fill placement in order to document the locations of excavation and/or to supervise reinstatement of the excavated areas to engineered fill status. If construction on the engineered fill does not commence within a period of 2 years from the date of certification, the condition of the engineered fill must be assessed for recertification.
13. Despite stringent control in the placement of the engineered fill, variations in soil type and density may occur in the engineered fill. The total and differential settlements of 25 mm and 15 mm, respectively, should be considered in the design of the foundations. They must be properly reinforced and designed by structural engineer for the project.

6.2 **Foundations**

6.2.1 Townhouse Blocks (Boreholes 1, 2 and 3)

The townhouse blocks will be 3-storey framed structures with basement. They can be supported on conventional footings founded in undisturbed native subsoil or on engineered fill below 1.5 to 2.0 m from the existing ground level. The recommended Maximum Allowable Soil Pressure (SLS) and the Factored Ultimate Soil Bearing Pressure (ULS) are presented below:

- Maximum Allowable Soil Bearing Pressure (SLS) = 150 kPa
- Factored Ultimate Soil Bearing Pressure (ULS) = 240 kPa

The total and differential settlements of the footings, designing for the bearing pressure at SLS, are estimated to be 25 mm and 15 mm, respectively.

6.2.2 **Mid-Rise Mixed Use Building** (Boreholes 4 and 5)

The mid-rise building will consist of 1-level underground parking. Assuming the groundwater is allowed to drain into the municipal sewer or a storage cistern will be designed to collect and store up the subsurface water for irrigation or surface cleaning during the dry season, the structure can be designed on conventional footings and the foundation depth is assumed below 3.5 m from grade.

Upon excavation to the foundation level, the subsoil is anticipated to consist of compact sand or silt. The recommended design bearing pressures for conventional footings are presented below:

- Maximum Allowable Soil Bearing Pressure (SLS) = 150 kPa
- Factored Ultimate Soil Bearing Pressure (ULS) = 250 kPa

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

Higher design bearing pressures of 500 kPa (SLS) will be available for the design of footings at a deeper level below 6 or 7 m from grade. Building foundations designing for the higher bearing pressures will have to be reviewed by the geotechnical engineer once the design is finalized.

Alternatively, the proposed building can be supported on helical piles extending into the dense stratum below 6 to 7 m from the prevailing ground surface. The load carried by each pile is directly related to the installation torque of the pile anchor in the underlying soil stratum. The founding elevations and the pile capacity should be determined by the prospective helical pile contractor.

If groundwater is not allowed to drain into the municipal sewer and a storage cistern is not favourable, the underground structure should be designed as a “tank” with a raft foundation to resist the hydrostatic pressure. Assumed the founding depth of 3.5 to 4 m from grade, the recommended design bearing pressures for raft foundation are presented below:

- Maximum Allowable Soil Bearing Pressure (SLS) = 200 kPa
- Factored Ultimate Soil Bearing Pressure (ULS) = 300 kPa

The total and differential settlements of the raft, designing for the bearing pressure at SLS, are estimated to be 25 mm and 20 mm, respectively. A Modulus of Subgrade Reaction of 20 MPa/m can be used for the design of the raft foundation.

6.2.3 Foundation Considerations

Foundations and grade beams exposed to weathering or in unheated areas, such as the exterior wall footings, footings near ventilation shaft and ramp-down driveway, should have at least 1.6 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. However, footings adjacent to the fresh air shafts, the entrance of the garage and any other areas which may be exposed to the outside temperature, a minimum frost cover of 1.6 m or equivalent insulation should be provided.

During construction, the footing subgrade and the installation of helical piles must 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.

A concrete mud-slab should be placed beneath the raft foundation immediately after exposure and inspection. If groundwater seepage is encountered in footing excavations, or where the subgrade is found to be wet, the footing subgrade should also be protected by a concrete mud-slab immediately after exposure. This will prevent construction disturbance and costly rectification.

The foundation design should meet the requirements specified in the latest Ontario Building Code. The structure should be designed to resist an earthquake force using Site Classification ‘D’ (stiff soil).

6.3 **Underground Structures**

In conventional construction of underground structures or basement, the perimeter walls should be dampproofed and provided with a perimeter subdrain (Drawing No. 3). Backfill of open excavation should consist of free-draining granular material unless prefabricated drainage board is installed over the entire wall below grade (Drawing No. 4). Underfloor weepers (Drawing No. 5) are also necessary where the subgrade is consisting of saturated soils. The subdrains and weepers should be shielded by a fabric filter and covered with stone filter to prevent blockage by silting, installed on a positive gradient and discharge into a positive outlet.

If the Municipality does not allow continuous discharge of groundwater into the municipal system, a storage cistern will be required to collect and store up the subsurface water for irrigation or surface cleaning during the dry season. If on-site cistern is not practical, the underground parking structure has to be waterproofed and designed to resist the hydrostatic pressure on the walls and the foundation. The elevator pit, which normally extends a few metres below the floor level, should also be designed as a submerged ‘tank’ structure with waterproofed pit walls and pit floor.

The perimeter walls of the underground structures should be designed to sustain a lateral earth pressure calculated using the soil parameters stated in Section 6.8 and any applicable surcharge loads adjacent to the proposed buildings must also be considered in the design of the foundation walls.

6.4 Slab-On-Grade and Concrete Sidewalk

The slab subgrade should either consist of native subsoils or engineered fill. A 20 cm thick granular base, consisting of 20-mm Crusher-Run Limestone or equivalent, compacted to its maximum Standard Proctor dry density, is required as a moisture barrier for slab-on-grade construction.

Prior to the placement of the granular base, the subgrade should be proof-rolled and inspected. Where soft subgrade is detected, it should be subexcavated and replaced with inorganic material compacted to 98% or + of its Standard Proctor dry density.

For a water-proofed underground structure with a raft foundation, the slab-on-grade will be poured on a granular fill above the concrete raft where the underground utilities and pipes will be laid.

The final grading around the buildings must be graded to direct water away from the structures to minimize the frost heave phenomenon generally associated with the disclosed soils.

To prevent frost action induced by cold wintry drafts in areas where vertical ground movement cannot be tolerated, such as building entrances, the subgrade material should be replaced with 1.6 m of non-frost-susceptible granular material such as Granular ‘B’ and provided with subdrains connected to the storm system. This will minimize frost action in this area where vertical ground movement cannot be tolerated. Alternatively, the sidewalk should be insulated with 50-mm Styrofoam, or equivalent.

6.5 Underground Services

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

The pipe joints must be leak-proof, or the joints should be wrapped with a waterproof membrane to prevent subgrade upfiltration through the joints. A Class 'B' bedding, consist of compacted 20-mm Crusher-Run Limestone, or equivalent. It is recommended for the underground service construction. In areas where saturated soil subgrade is evident, a Class 'A' concrete bedding should be used.

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

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

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.

Backfill in trenches and excavated areas should be compacted to at least 95% of its maximum Standard Proctor dry density. Below any floor slab sensitive to settlement and in the zone within 1.0 m below the pavement subgrade, the earth fill must be compacted to at least 98% of its maximum Standard Proctor dry density.

In normal underground services construction practice, the problem areas of settlement largely occur adjacent to manholes, catch basins, services crossings, foundation walls and columns. In areas which are inaccessible to a heavy compactor, imported 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 for services crossings 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 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 flooding 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 and the slab-on-grade construction.
- To backfill a deep trench, one must be aware that future settlement is to be expected, unless the side of the cut is flattened to at least 1 vertical: 1.5+ horizontal, and the lifts of the fill and its moisture content are stringently controlled; i.e., lifts should be no more than 20 cm (or less if the backfilling conditions dictate) and uniformly compacted to achieve at least 95% of the maximum Standard Proctor dry density, with the moisture content on the wet side of the optimum.
- It is often difficult to achieve uniform compaction of the backfill in the lower vertical section of a trench which is an open cut or is stabilized by a trench box, particularly in the sector close to the trench walls or the sides of the box. These sectors must be backfilled with sand. In a trench stabilized by a trench box, the void left after the removal of the box will be filled by the backfill. It is necessary to backfill this sector with sand, and the compacted backfill must be flooded for 1 day, prior to the placement of the backfill above this sector; i.e., in the upper sloped trench section. This measure is necessary in order to prevent consolidation of inadvertent voids and loose backfill which will compromise the compaction of the backfill in the upper section. In areas where groundwater movement is expected in the sand fill mantle, anti-seepage collars should be provided.

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 on top of the underground garage is presented in Table 3.

Table 3 - Pavement Design (on Roof of Underground Garage)

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	50	HL-8
Granular Base	250	OPSS Granular A or equivalent
Granular Sub-base	100	Free-Draining Sand Fill

The recommended pavement structure for on-grade parking and driveway is given in Table 4.

Table 4 - Pavement Design (On-Grade Parking and Driveway)

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder Light Duty Car Park Driveway/ Fire Route	 50 65	HL-8
Granular Base	150	OPSS Granular A or equivalent
Granular Sub-base Light Duty Car Park Driveway/ Fire Route	 250 350	OPSS Granular B or equivalent

The fine graded subgrade surface should be proof-rolled and any soft spot as identified should be subexcavated and replaced by properly compacted inorganic earth fill. In low lying areas and along the perimeter where runoff may drain onto the pavement, 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). Subdrains consisting of filter-sleeved weepers should be installed at 0.3 m below the underside of the pavement structure, 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.

The earth fill within the zone of 1.0 m below the pavement must be compacted to 98% or + of its maximum Standard Proctor dry density, with the moisture content 2% to 3% drier than the optimum. In the lower zone, a 95% or + Standard Proctor compaction is considered adequate. The granular bases should be compacted to 100% of their maximum Standard Proctor dry density.

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

- If pavement construction does not immediately follow the trench backfilling, the subgrade should be properly crowned and smooth-rolled to allow interim precipitation to be properly drained.
- The areas adjacent to the pavement should be properly graded to prevent the ponding of large amounts of water during the interim construction period.
- If the pavement is to be constructed during the wet seasons and extensively soft subgrade occurs, the granular sub-base may require thickening. This can be assessed during construction.

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/m^3)**	**Estimated Bulk Factor**	
	Bulk	**Loose**	**Compacted**
Earth Fill	21.0	1.20	0.98
Sand and Silt	21.5	1.25	1.00
Silty Clay	22.0	1.30	1.03
Lateral Earth Pressure Coefficients	**Active K_a**	**At Rest K_o**	**Passive K_p**
Compacted Earth Fill	0.40	0.55	2.50
Sand and Silt	0.35	0.50	3.00
Silty Clay	0.45	0.60	2.20

6.9 Excavation

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

Table 6 - Classification of Soils for Excavation

Material	Type
Earth Fill, Silty Clay, Drained Sand and Silt	3
Saturated Soils	4

All excavation must be sloped back for safety. Temporary shoring will be required in excavation where a safe backing slope cannot be provided due to space limitation. The design parameters for the shoring and our recommendations are provided in the Appendix. The overburden load and the surcharge from any adjacent structures should also be considered in the design of the shoring.

The groundwater recorded in the Monitoring Wells at Boreholes 2 and 3 ranged from 255.75 to 256.45 m, or slightly more than 4 m below the prevailing ground surface. The remaining boreholes are mostly dry within the depth of investigation. In excavation, the yield of groundwater is anticipated to be localized and in limited quantities.

Where excavations are carried out in saturated soils, the possibility of flowing sides and bottom boiling dictates that the ground should be predrained by pumping from closely spaced sump-wells and/or the use of a well-point dewatering system.

One must be aware that the dewatering and excavation for the underground structure may have an impact on the adjacent properties. Preconstruction survey and an appropriate monitoring programme of the adjacent properties will be required in order to ensure the existing structures and foundations are not compromised.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of 2596843 Ontario Inc., and for review by designated consultants and government agencies. Use of the report is subject to the conditions and limitations of the contractual agreement.

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

SOIL ENGINEERS LTD.

Weida (Daric) Yang, B.A.Sc.

Bennett Sun, P.Eng.
WY/BS

LICENSED PROFESSIONAL ENGINEER
CHUN C. B. SUN
June 6, 2019
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.: 1802-S072

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

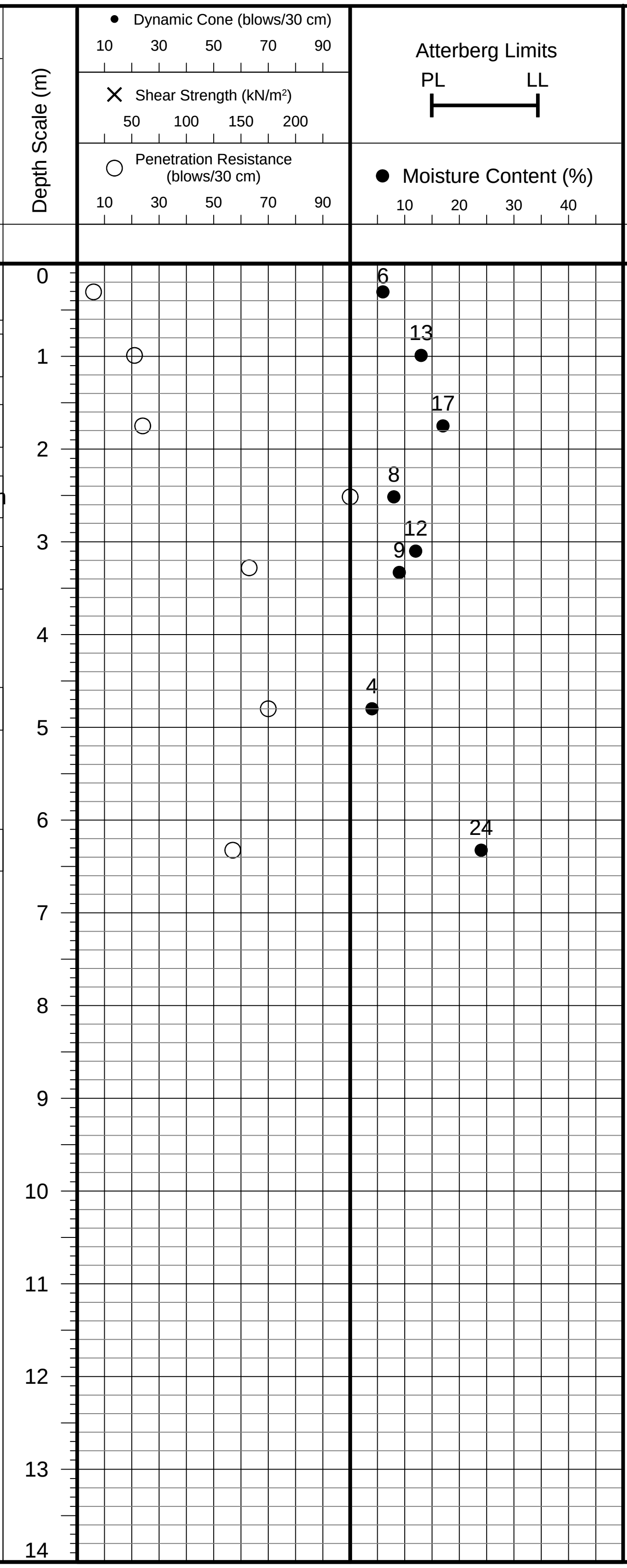
PROJECT DESCRIPTION: Proposed 3-Storey Mixed Use Building with 1-Level Underground Parking

PROJECT LOCATION: 224 Ardagh Road, City of Barrie

METHOD OF BORING: Flight-Auger (Hollow-Stem)

DRILLING DATE: March 28, 2018

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
259.8	Ground Surface			
259.6 0.2	**21 cm TOPSOIL**			
	EARTH FILL brown to dark brown silty sand trace of organics	1	DO	6
258.8 1.0	Brown, compact to very dense **SAND** fine to medium grained some silt to silty trace of gravel	2 3 4	DO DO DO	21 24 50/3 cm
256.7 3.1	Brown, very dense **SILT** a trace of clay occ. seams of fine sand	5A 5B	AS	63
255.4 4.4	Brown, very dense **SAND** fine grained silty	6 7	DO DO	70 57
253.3 6.5	**END OF BOREHOLE**			



WATER LEVEL

Cave-in @ El. 254.0 m upon completion.

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JOB NO.: 1802-S072

LOG OF BOREHOLE NO.: 2

FIGURE NO.: 2

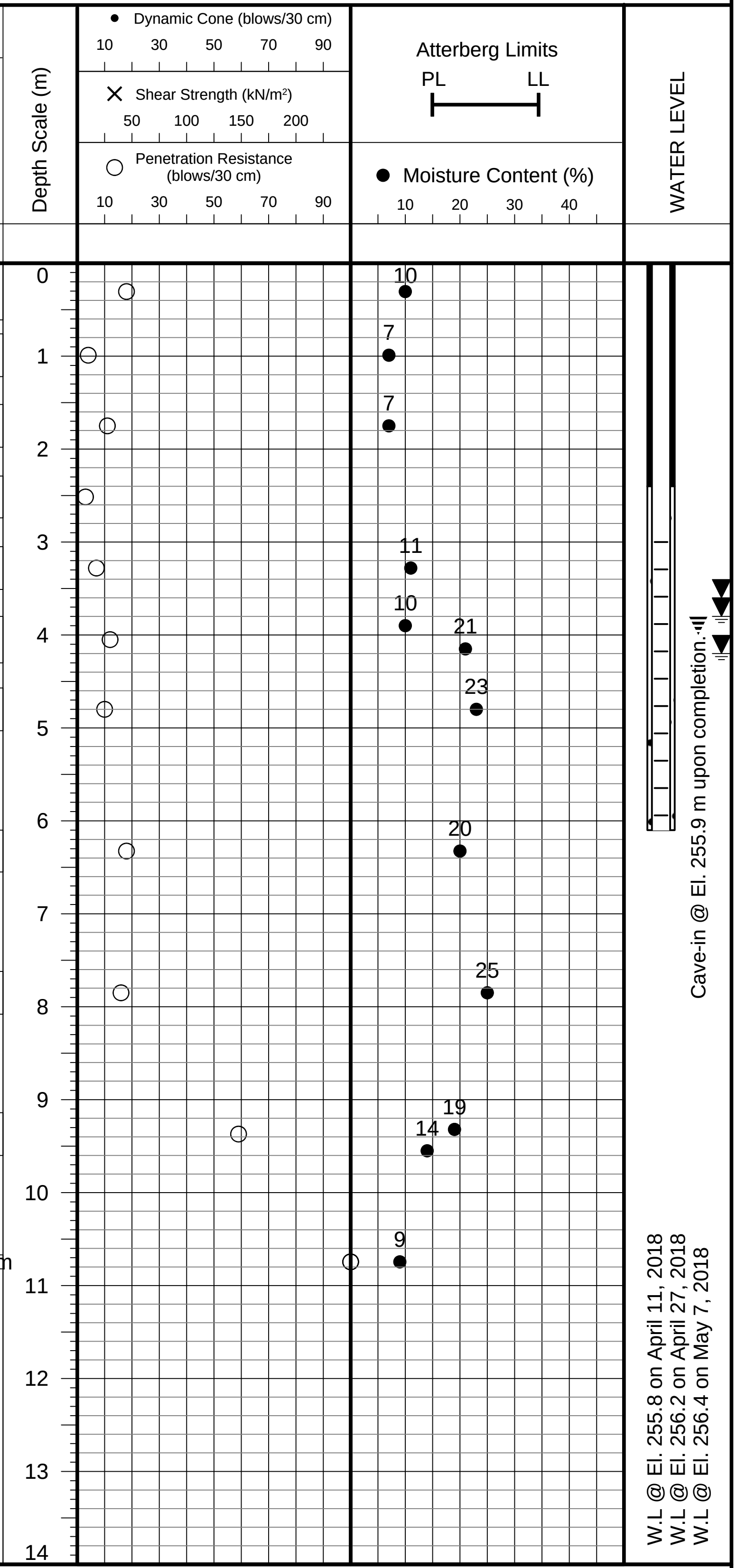
PROJECT DESCRIPTION: Proposed 3-Storey Mixed Use Building with 1-Level Underground Parking

PROJECT LOCATION: 224 Ardagh Road, City of Barrie

METHOD OF BORING: Flight-Auger (Hollow-Stem)

DRILLING DATE: March 28, 2018

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
259.9	Ground Surface			
0.0 259.3	**8 cm ASPHALT** **530 mm GRANULAR FILL**	1	DO	18
0.6	**EARTH FILL** brown silty sand concrete rubble below 1.6 m	2	DO	4
		3	DO	11
		4	DO	3
		5	DO	7
		6A 6B	DO	12
		7	DO	10
255.0 4.9	Brown, compact to dense **SILT** some clay seams of fine sand and silty clay occ. layers of silty clay below 7.0 m	8	DO	18
		9	DO	16
		10A 10B	DO	59
250.4 9.5	Brown, very dense **SAND** fine grained silty	11	DO	55/15 cm
249.1 10.8	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 6.1 m Slotted screen from 3.1 to 6.1 m Sand backfill from 2.4 to 6.1 m Bentonite seal from 0 to 2.4 m Provided with a protective monument casing			



Cave-in @ El. 255.9 m upon completion.

W.L @ El. 255.8 on April 11, 2018
W.L @ El. 256.2 on April 27, 2018
W.L @ El. 256.4 on May 7, 2018

Soil Engineers Ltd.

JOB NO.: 1802-S072

LOG OF BOREHOLE NO.: 3

FIGURE NO.: 3

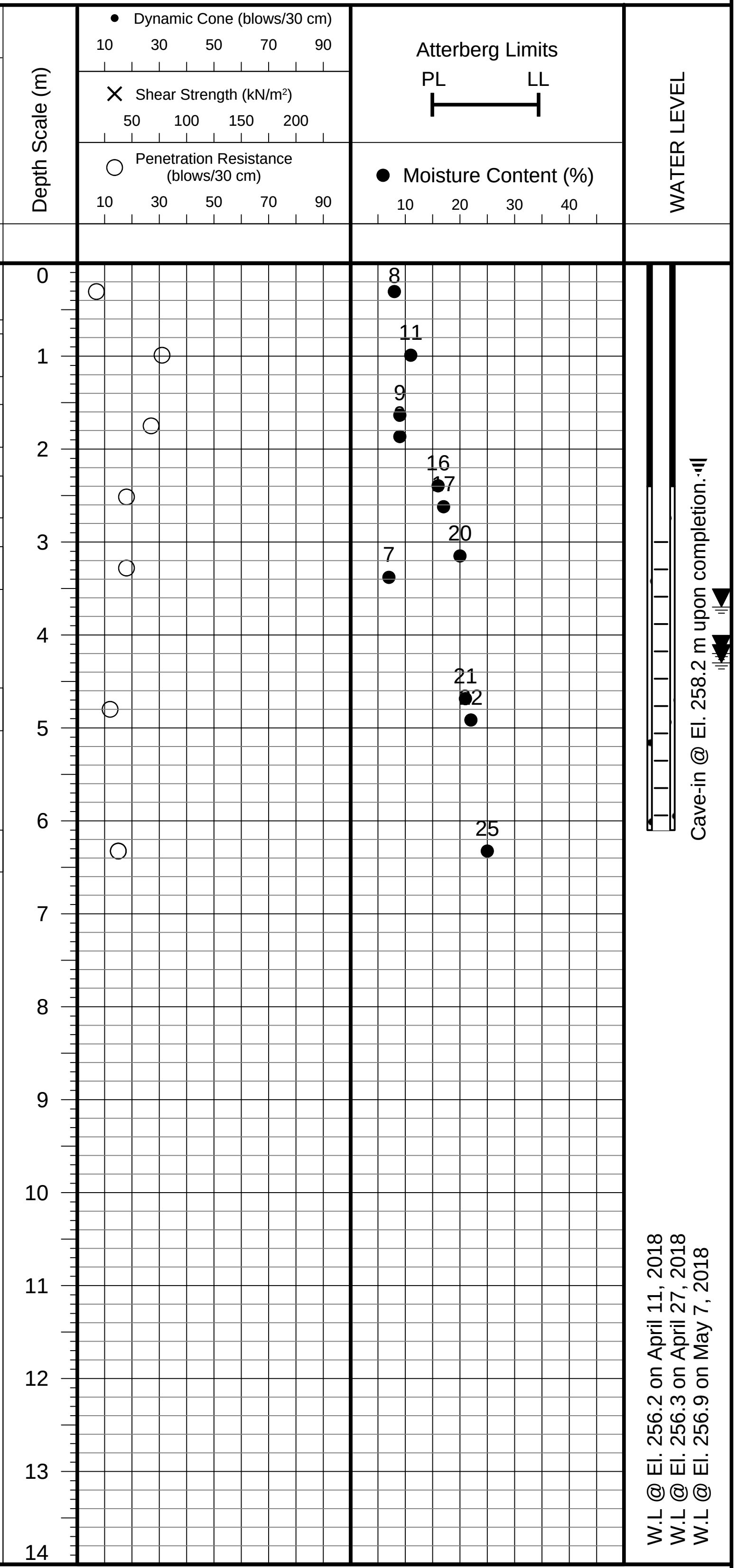
PROJECT DESCRIPTION: Proposed 3-Storey Mixed Use Building with 1-Level Underground Parking

METHOD OF BORING: Flight-Auger (Hollow-Stem)

PROJECT LOCATION: 224 Ardagh Road, City of Barrie

DRILLING DATE: March 28, 2018

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
260.5 0.0	Ground Surface **16 cmTOPSOIL**			
	Brown, loose to dense **SAND** fine to medium grained some silt to silty occ. layers os sandy silt fine grained below 1.6 m	1	DO	7
		2	DO	31
		3A 3B	DO	27
		4A 4B	DO	18
		5A 5B	DO	18
255.7 4.8	Brown, stiff **SILTY CLAY** seams of fine sand and silt	6A 6B	DO	12
		7	DO	15
253.9 6.5	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 6.1 m Slotted screen from 3.1 to 6.1 m Sand backfill from 2.4 to 6.1 m Bentonite seal from 0 to 2.4 m Provided with a protective monument casing			



Cave-in @ El. 258.2 m upon completion.

W.L @ El. 256.2 on April 11, 2018
W.L @ El. 256.3 on April 27, 2018
W.L @ El. 256.9 on May 7, 2018

JOB NO.: 1802-S072

LOG OF BOREHOLE NO.: 4

FIGURE NO.: 4

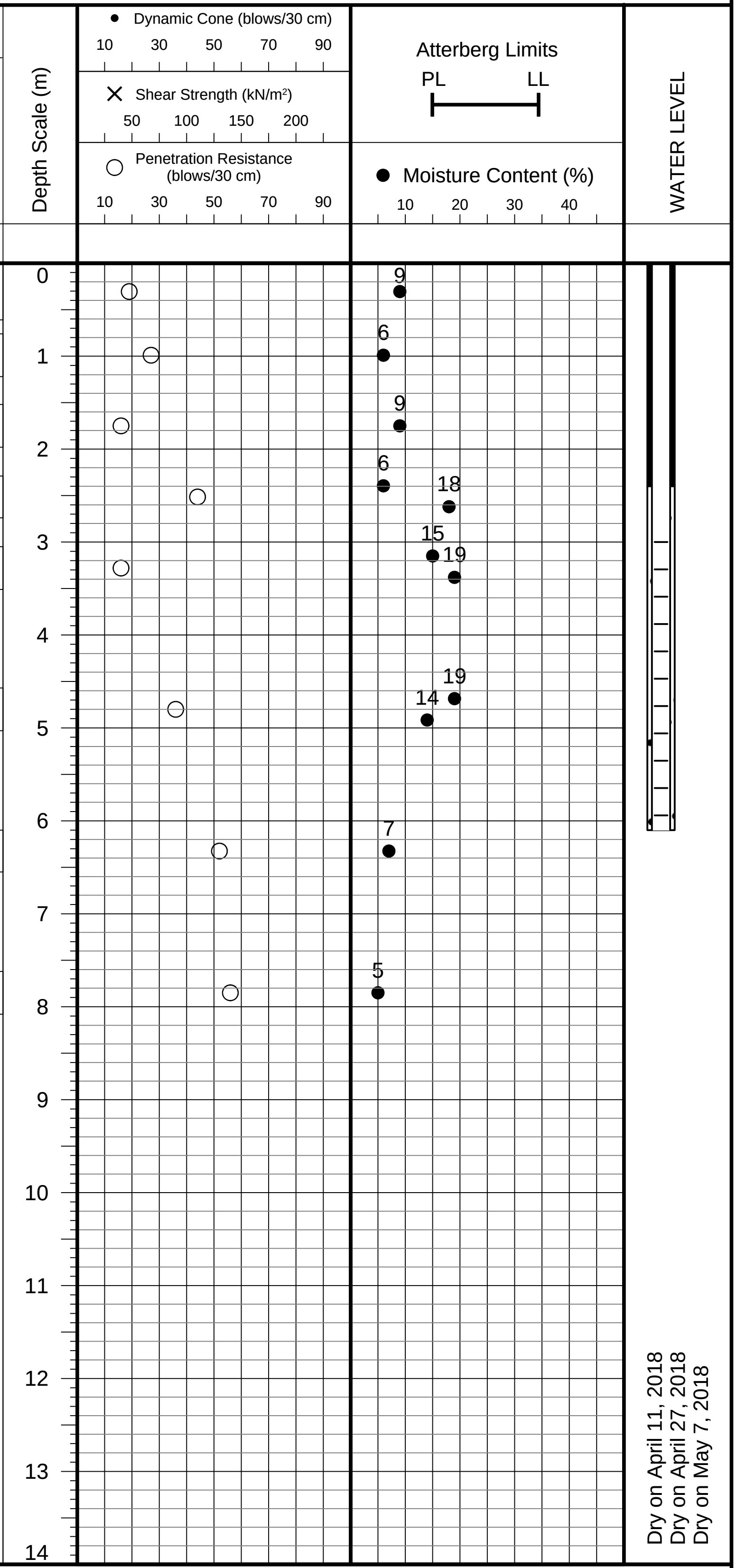
PROJECT DESCRIPTION: Proposed 3-Storey Mixed Use Building with 1-Level Underground Parking

PROJECT LOCATION: 224 Ardagh Road, City of Barrie

METHOD OF BORING: Flight-Auger (Hollow-Stem)

DRILLING DATE: March 28, 2018

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
261.2	Ground Surface			
0.0	**21 cm TOPSOIL** Brown, compact to dense **SAND** fine to medium grained occ. layers of sandy silt	1	DO	19
		2	DO	27
		3	DO	16
258.7 2.5	Brown, compact to dense **SILT** trace of clay seams of fine sand	4A 4B	DO	44
		5A 5B	DO	16
256.4 4.8	Brown, dense to very dense **SAND** fine grained silty seams of silt	6A 6B	DO	36
		7	DO	52
		8	DO	56
253.1 8.1	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 6.1 m Slotted screen from 3.1 to 6.1 m Sand backfill from 2.4 to 6.1 m Bentonite seal from 0 to 2.4 m Provided with a protective monument casing			



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JOB NO.: 1802-S072

LOG OF BOREHOLE NO.: 5

FIGURE NO.: 5

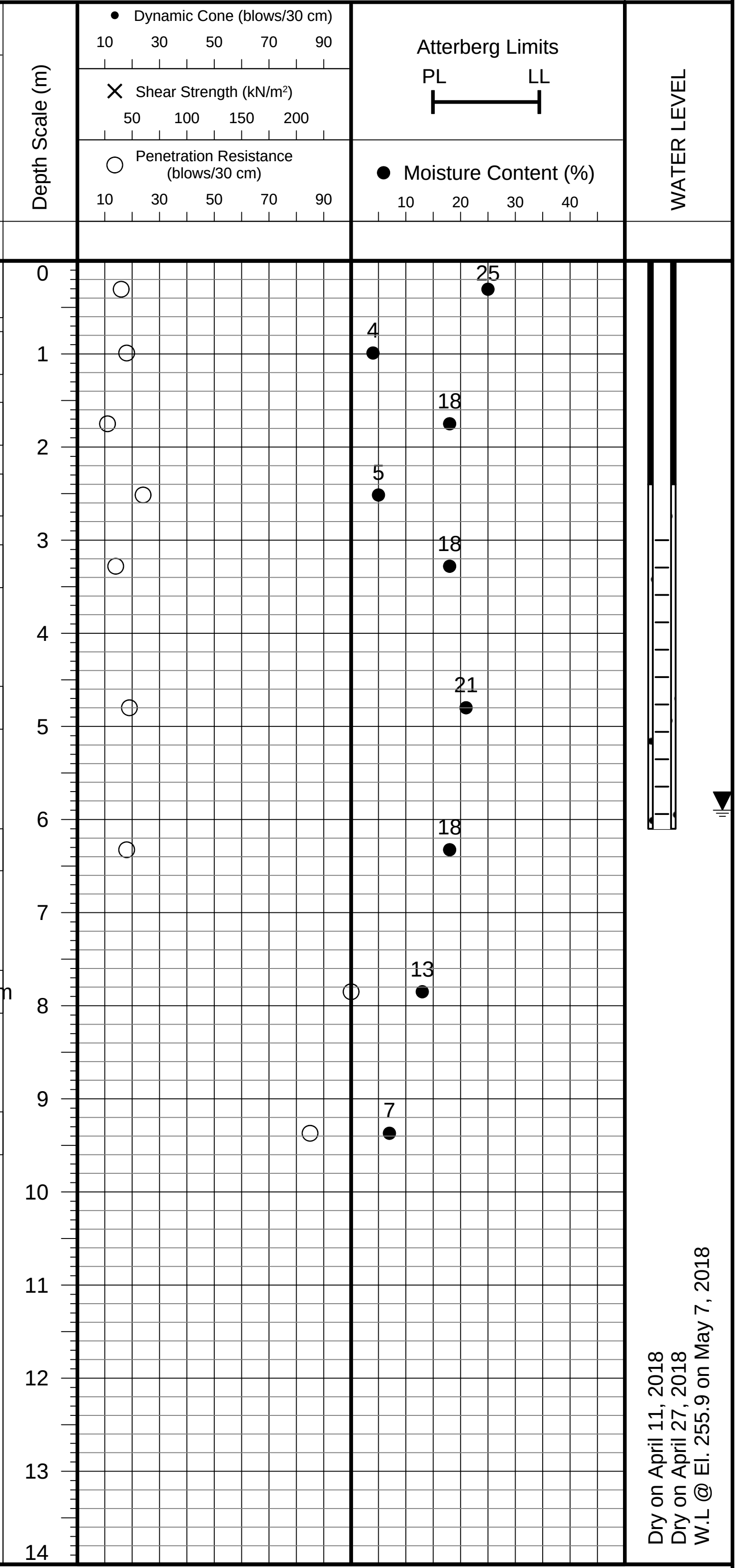
PROJECT DESCRIPTION: Proposed 3-Storey Mixed Use Building with 1-Level Underground Parking

PROJECT LOCATION: 224 Ardagh Road, City of Barrie

METHOD OF BORING: Flight-Auger (Hollow-Stem)

DRILLING DATE: March 28, 2018

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
261.8	Ground Surface			
0.0	**21 cm TOPSOIL** **EARTH FILL** dark brown sand and gravel some silt and rootlets	1	DO	16
261.2 0.6	Brown, compact **SAND** fine to medium grained some silt to silty weathered to 2.0 m	2	DO	18
		3	DO	11
		4	DO	24
		5	DO	14
257.5 4.3	Brown, compact **SILT** some clay occ. sand seams	6	DO	19
		7	DO	18
254.5 7.3	Brown, very dense **SAND** fine grained silty occ. cobbles	8	DO	50/15 cm
		9	DO	85
252.2 9.6	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 6.1 m Slotted screen from 3.1 to 6.1 m Sand backfill from 2.4 to 6.1 m Bentonite seal from 0 to 2.4 m Provided with a protective monument casing			



Dry on April 11, 2018
Dry on April 27, 2018
W.L @ El. 255.9 on May 7, 2018

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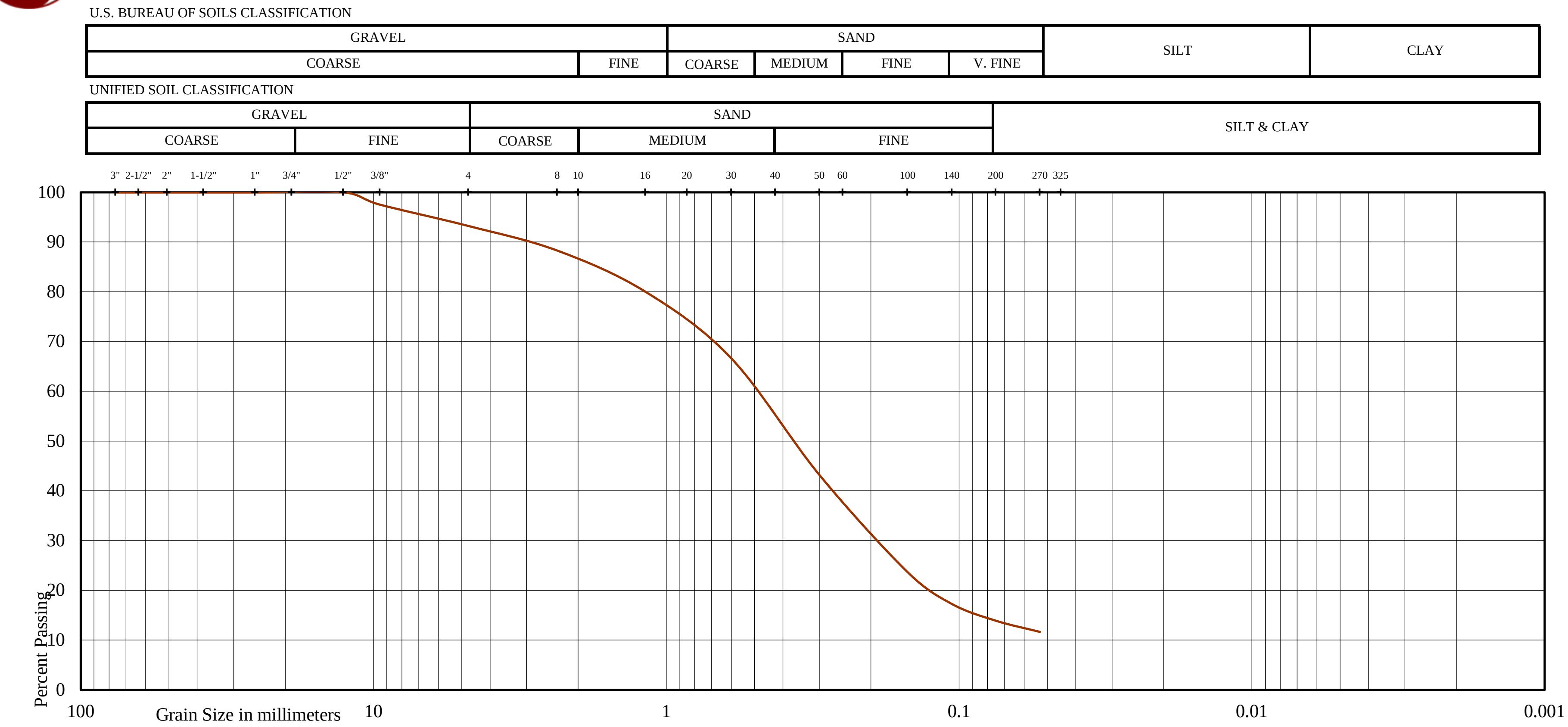
GRAIN SIZE DISTRIBUTION

Reference No: 1802-S072



Project:	Proposed 3-Storey Mixed Use Building with 1-Level Underground Parking			
Location:	224 Ardagh Road, City of Barrie		Liquid Limit (%) =	-
			Plastic Limit (%) =	-
Borehole No:	4		Plasticity Index (%) =	-
Sample No:	1		Moisture Content (%) =	9
Depth (m):	0.4		Estimated Permeability	
Elevation (m):	261.6		(cm./sec.) =	10^{-3}

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

Figure: 6

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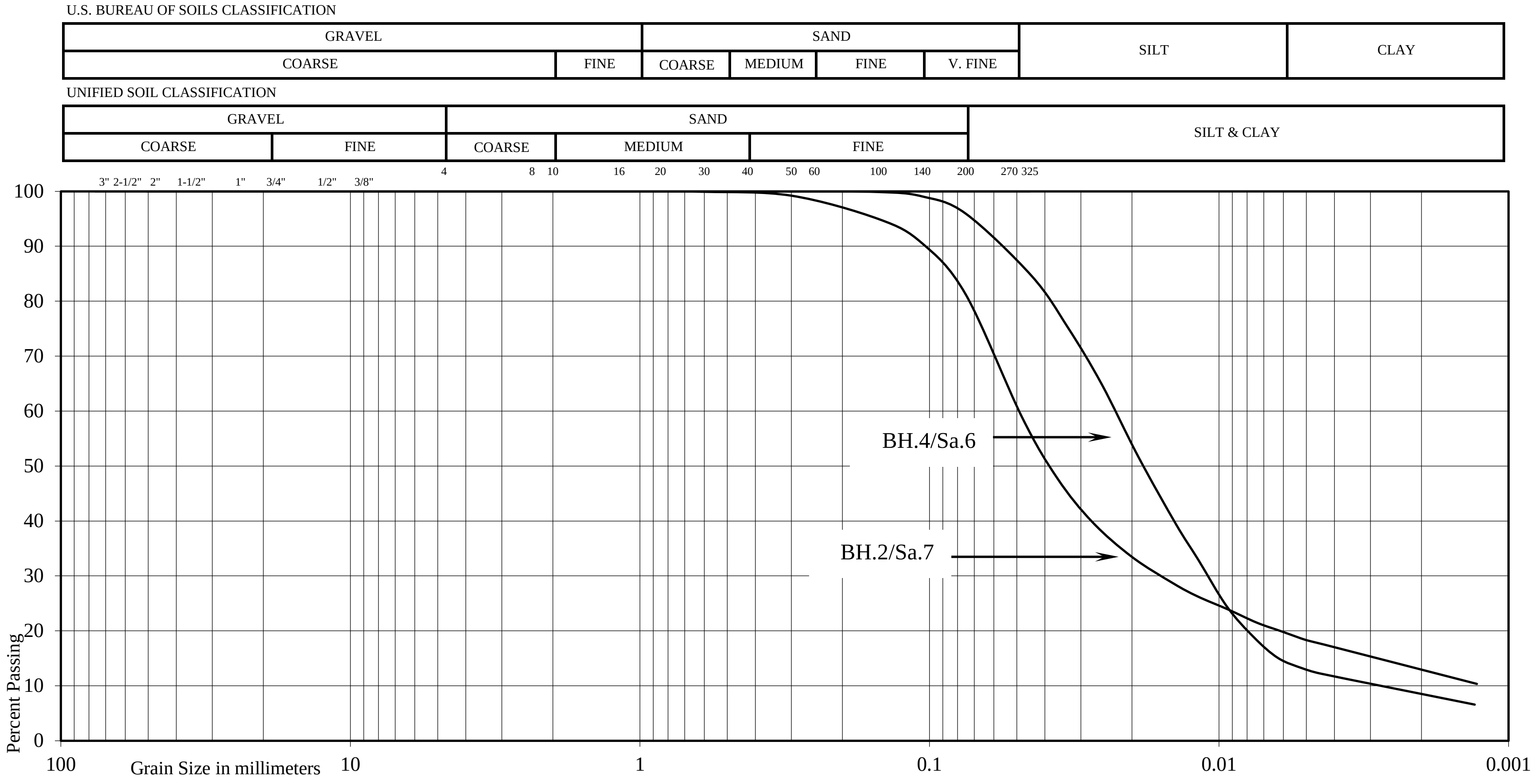
GRAIN SIZE DISTRIBUTION

Reference No: 1802-S072



Project: Proposed 3-Storey Mixed Use Building with 1-Level Underground Parking

Location: 224 Ardagh Road, City of Barrie

Borehole No:	2	4
Sample No:	7	6A
Depth (m):	4.8	4.6
Elevation (m):	255.5	257.4

BH./Sa.	2/7	4/6A
Liquid Limit (%) =	-	-
Plastic Limit (%) =	-	-
Plasticity Index (%) =	-	-
Moisture Content (%) =	-	-
Estimated Permeability (cm./sec.) =	10^{-3}	10^{-4}

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

Figure: 7

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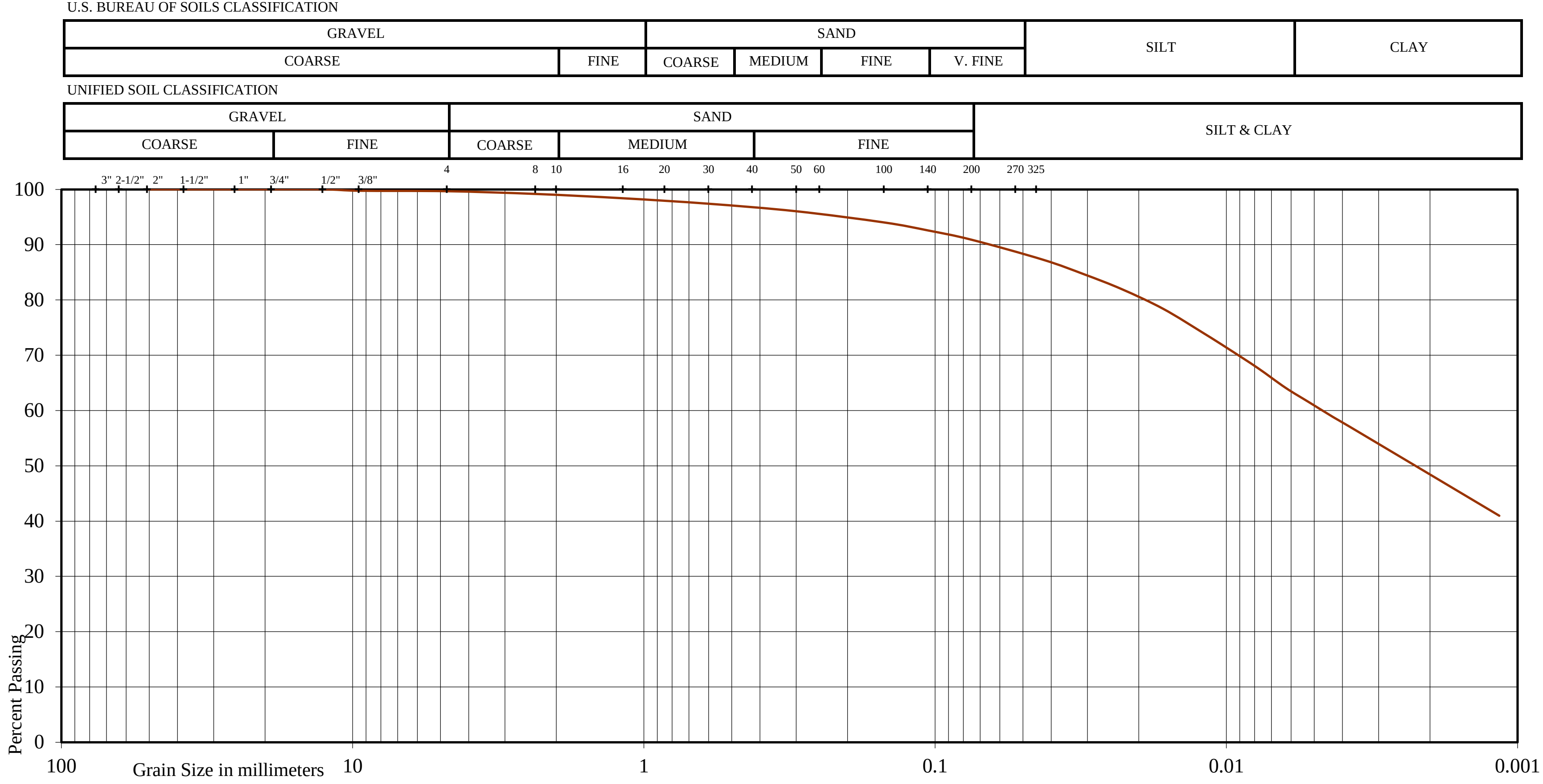
GRAIN SIZE DISTRIBUTION

Reference No: 1802-S072



Project: Proposed 3-Storey Mixed Use Building with 1-Level Underground Parking

Location: 224 Ardagh Road, City of Barrie

Borehole No: 3

Sample No: 7

Depth (m): 6.3

Elevation (m): 254.8

Liquid Limit (%) = 45

Plastic Limit (%) = 23

Plasticity Index (%) = 22

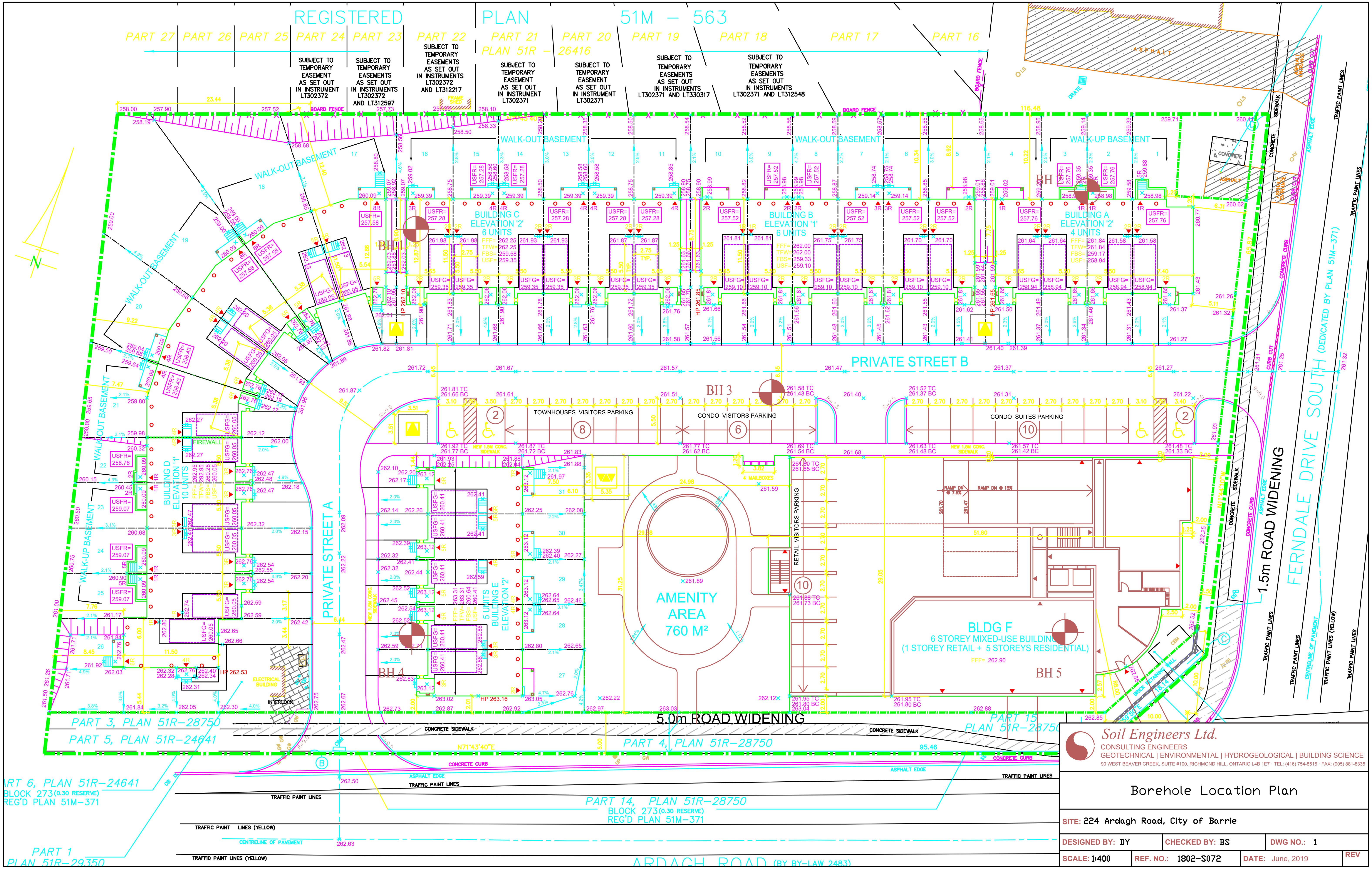
Moisture Content (%) = 25

Estimated Permeability

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

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

Figure: 8



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Borehole Location Plan

SITE: 224 Ardagh Road, City of Barrie

DESIGNED BY: DY | CHECKED BY: BS | DWG NO.: 1

SCALE: 1:400 | REF. NO.: 1802-S072 | DATE: June, 2019 | REV

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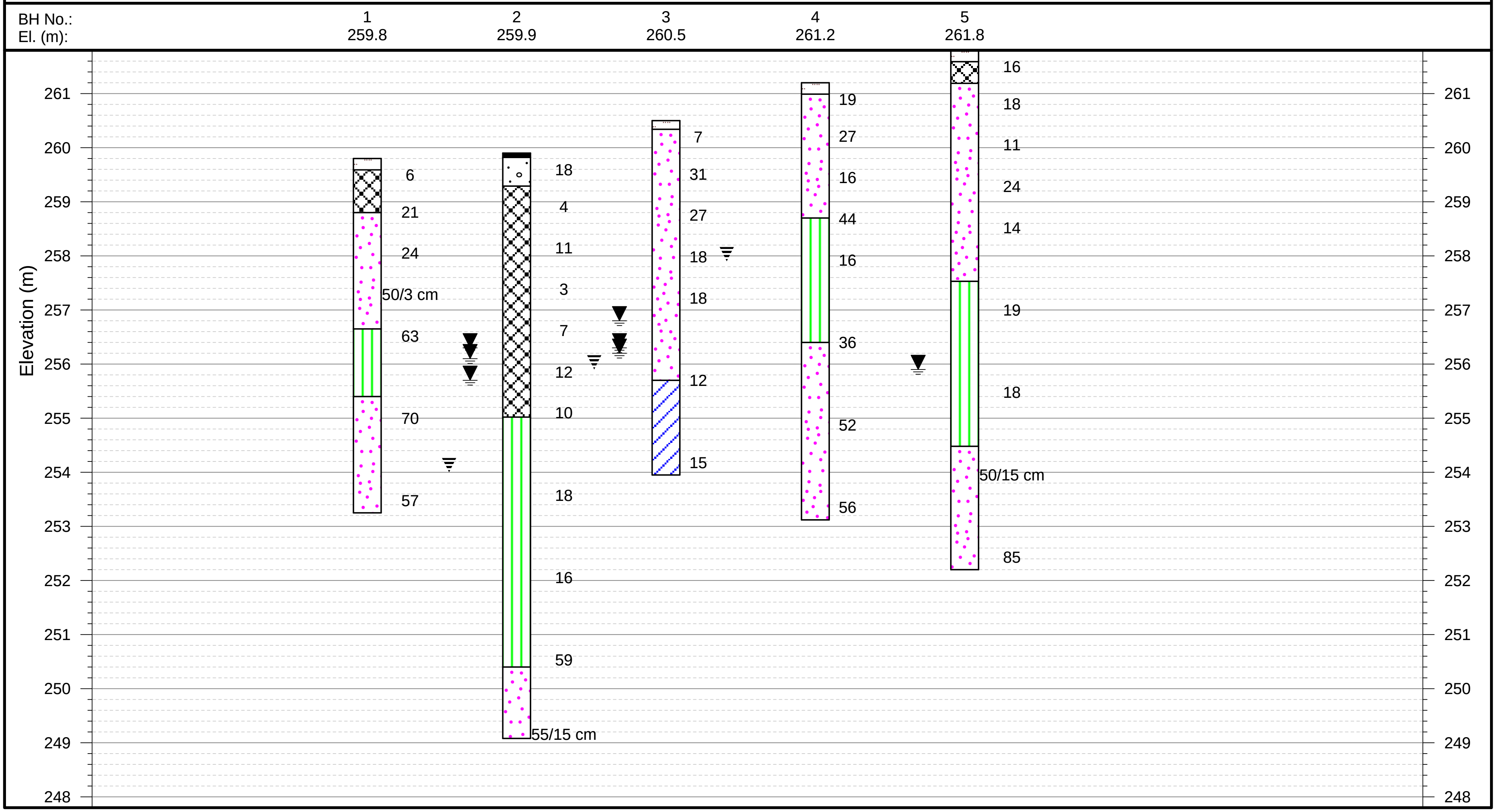
SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

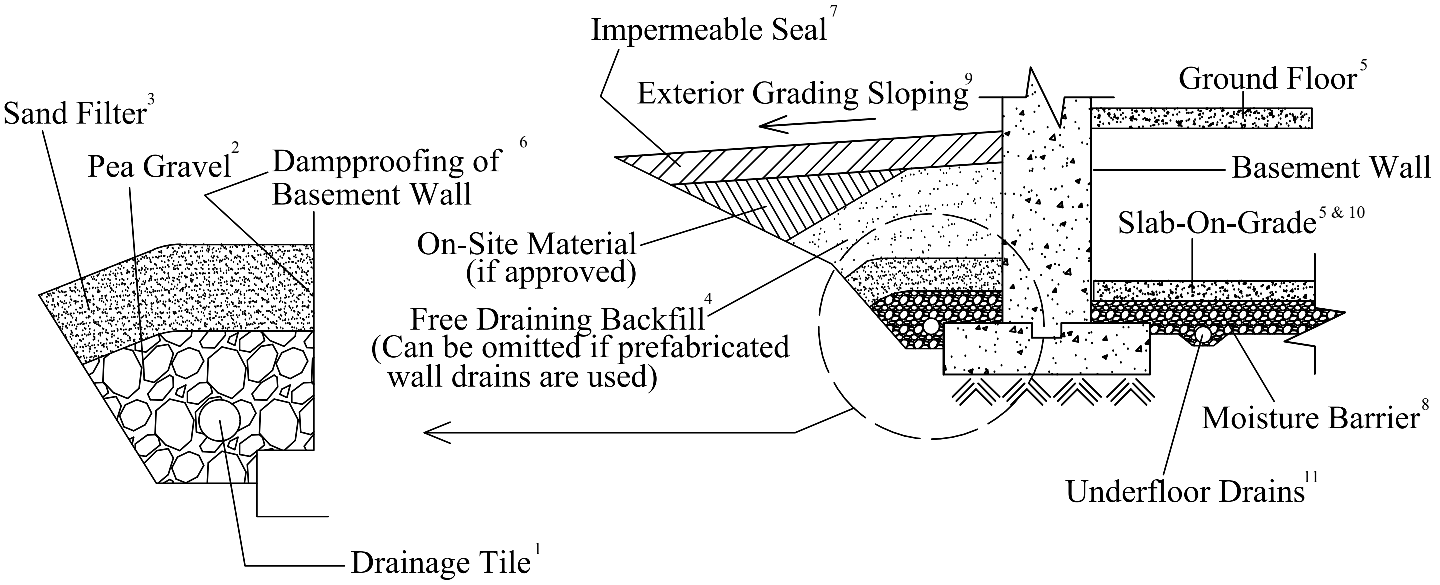
JOB NO.: 1802-S072
REPORT DATE: June 2019
PROJECT DESCRIPTION: Proposed 3-Storey Mixed Use Building with 1-Level Underground Parking
PROJECT LOCATION: 224 Ardagh Road, City of Barrie

LEGEND

▨ TOPSOIL
■ ASPHALT
▨ GRANULAR
▨ FILL
▨ SAND
▨ SILT
▨ SILTY CLAY

≡ CAVE-IN
▼ WATER LEVEL (STABILIZED)





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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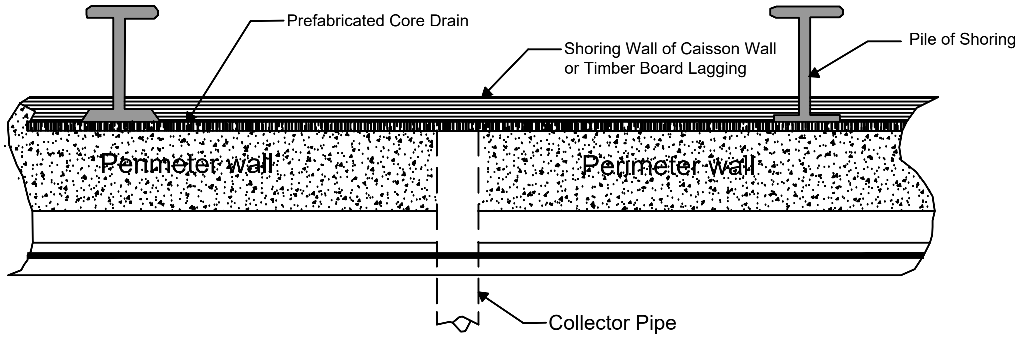
Perimeter Drainage

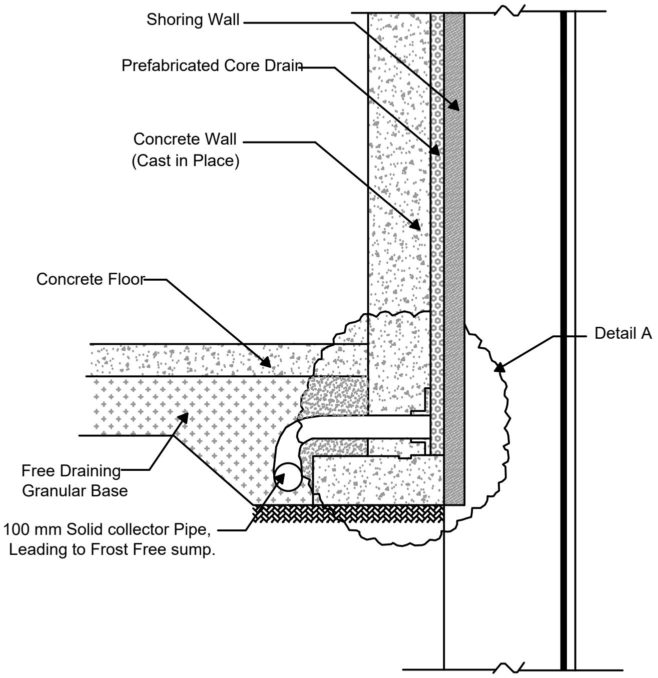
SITE 224 Ardagh Road, City of Barrie

DESIGNED BY D.Y. **CHECKED BY** B.S. **DWG NO.** 3

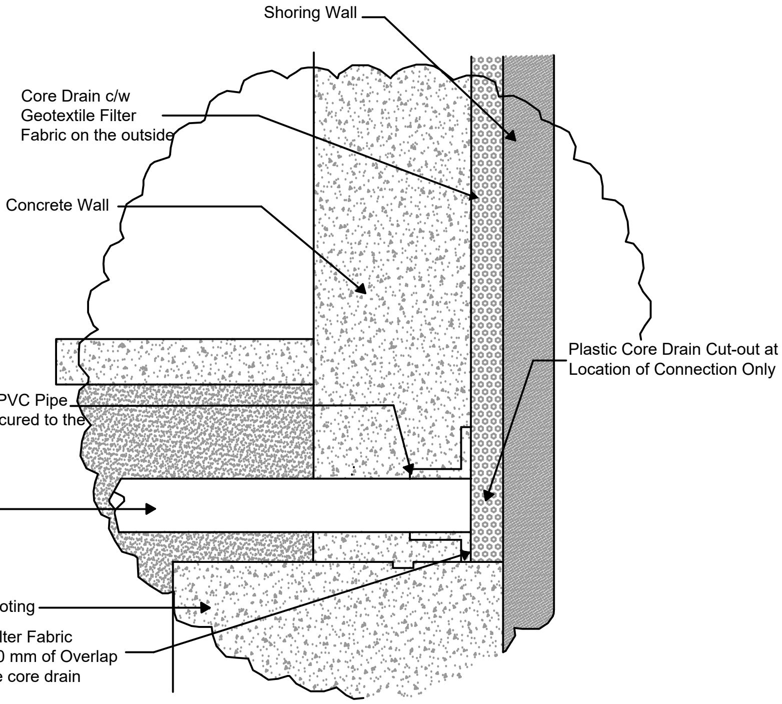
SCALE N.T.S. **REF. NO.** 1802-S072 **DATE** June, 2019 **REV** -



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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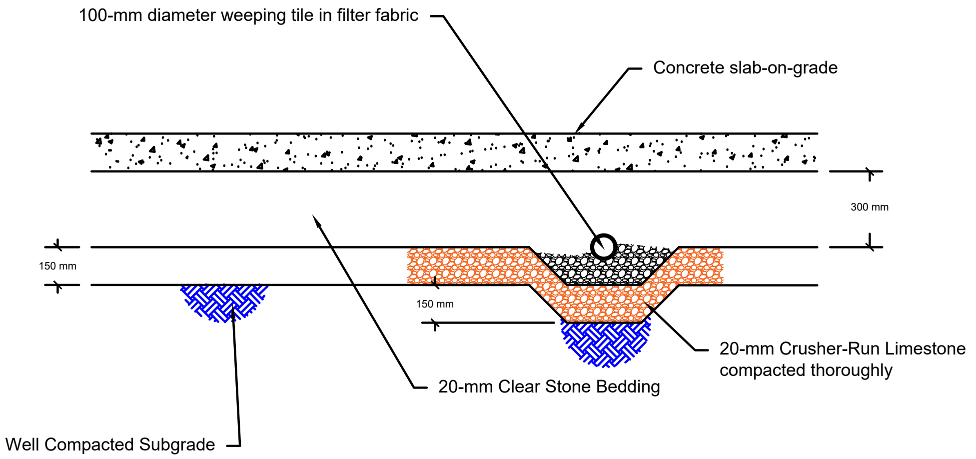
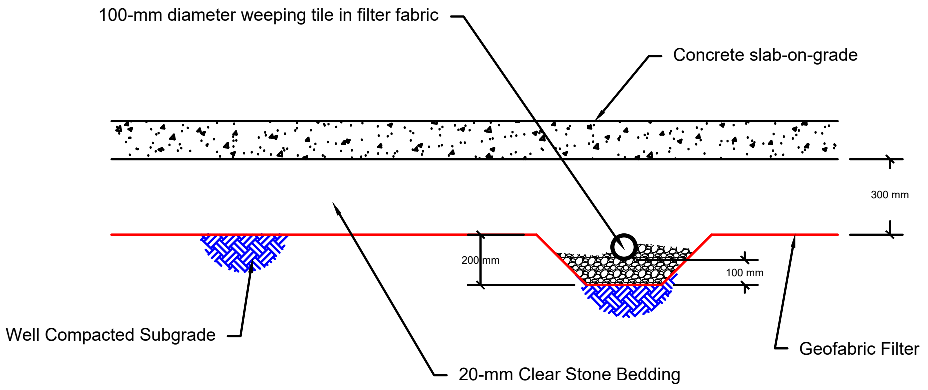
Perimeter Drainage System with Shoring

SITE: 224 Ardagh Road, City of Barrie

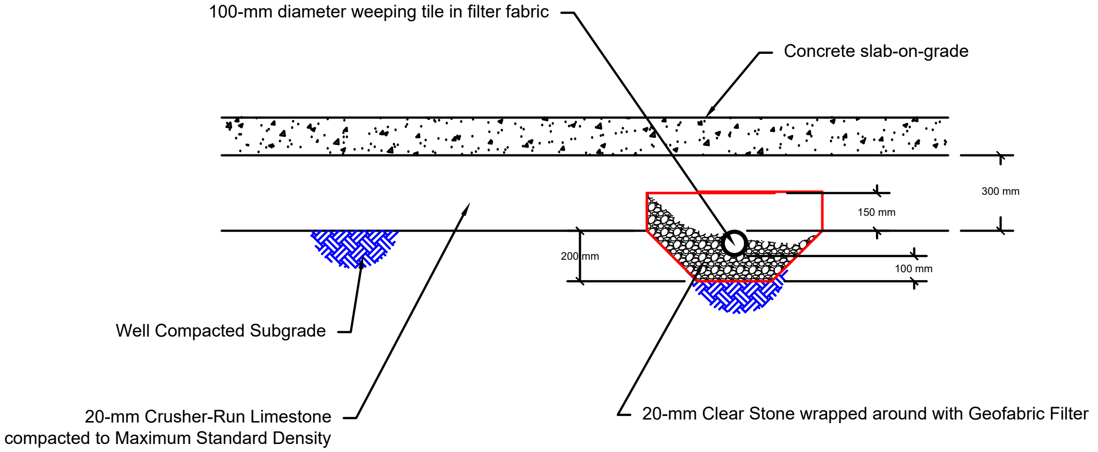
DESIGNED BY: D.Y. **CHECKED BY:** B.S. **DWG NO.:** 4
SCALE: N.T.S. **REF. NO.:** 1802-S072 **DATE:** June, 2019 **REV** -



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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Underfloor Weepers

SITE: 224 Ardagh Road, City of Barrie

DESIGNED BY: D.Y. **CHECKED BY:** B.S. **DWG NO.:** 5

SCALE: N.T.S. **REF. NO.:** 1802-S072 **DATE:** June, 2019 **REV** -

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APPENDIX

SHORING DESIGN

REFERENCE NO. 1802-S072

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 25 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 native soil deposit below the bottom of excavation should be designed for the allowable bearing pressure of 50 kPa (1.0 k.s.f.).

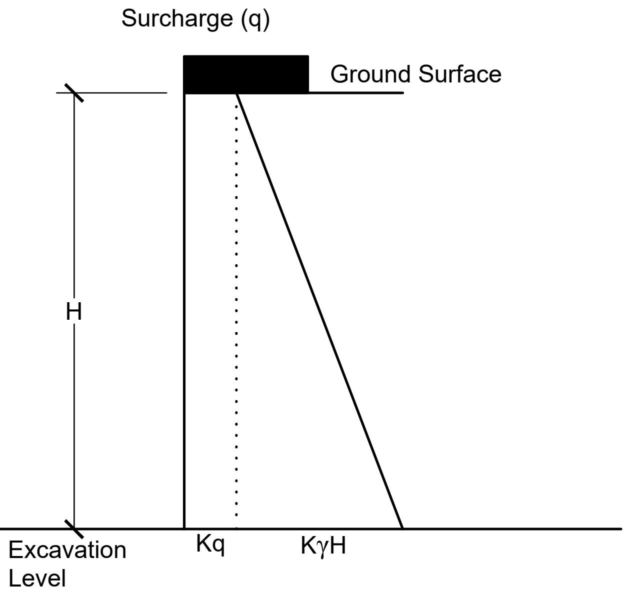
The raker footings should be located outside the zone of influence of the buried portion of the soldier piles at a distance of not less than 1.5 of the length of embedment of the soldier pile.

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.

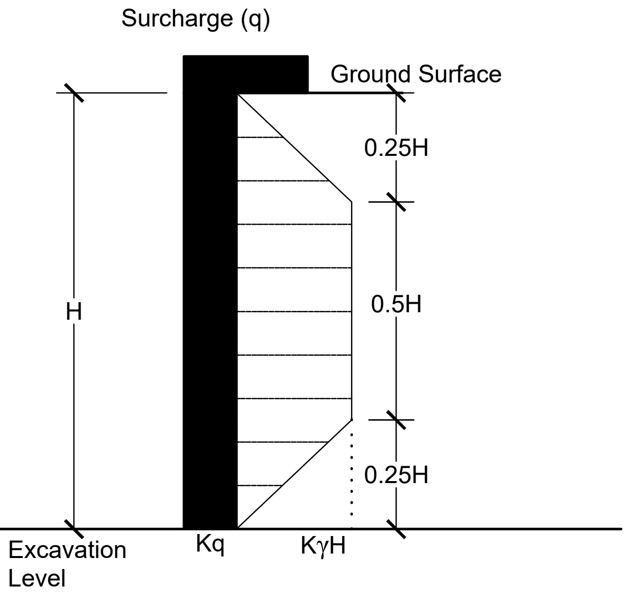
MONITORING OF PERFORMANCE

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.

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 21 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 - Earth Pressure Distribution

SITE: 224 Ardagh Road, City of Barrie

DESIGNED BY: D.Y. **CHECKED BY:** B.S. **DWG NO.:** A1

SCALE: N.T.S. **REF. NO.:** 1802-S072 **DATE:** June 2019 **REV** -