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A REPORT TO
BROCK ROAD DUFFINS FOREST INC.

A GEOTECHNICAL INVESTIGATION FOR PROPOSED RESIDENTIAL DEVELOPMENT

2055 BROCK ROAD

CITY OF PICKERING

REFERENCE NO. 1909-S140

DECEMBER 2019

DISTRIBUTION

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1.0 INTRODUCTION

In accordance with written authorization dated September 23, 2019, from Ms. Alison L. Lin, Project Director of Brock Road Duffins Forest Inc., a geotechnical investigation was carried out at the property located at 2055 Brock Road in the City of Pickering.

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 a proposed residential development. The geotechnical findings and resulting recommendations are presented in this Report.

2.0 SITE AND PROJECT DESCRIPTION

The City of Pickering is situated on Iroquois (glacial lake) plain where, in places, the glacial till stratigraphy has been partly eroded by the water action of the glacial lake and filled with lacustrine sands, silts, clays and reworked till, where it beds onto shale bedrock of Meaford-Dundas Formation at moderate to considerable depths.

The site, approximately 1.3 hectares (3.24 acres) in area, is located at the southeast quadrant of Brock Road and Usman Road in the City of Pickering. At the time of investigation, the site appeared to have been pregraded, with stockpiles of earth fill in place. The existing site gradient, despite the stockpiles, is relatively flat, with gravel and weed growth at the ground surface.

A review of the Architectural Plan indicates that the proposed development includes a residential building (Block A) of 6 and 20 storeys, connected by a 4-storey podium at the west portion. The east portion will consist of three structures (Blocks B, C and D) of two stacked townhouse blocks and one street townhouse block. These buildings will be constructed on an adjoined underground parking, with a lower parking level in the west portion.

3.0 FIELD WORK

The field work, consisting of ten (10) sampled boreholes and extending to depths of 12.0 to 12.7 m from the prevailing ground surface, was performed between October 11 and 21, 2019, at the locations shown on the Borehole Location Plan, Drawing No. 1.

The boreholes were advanced at intervals to the sampling depths by a track-mounted, continuous-flight power-auger machine equipped for soil sampling. Standard Penetration Tests, using the procedures described on the enclosed "List of Abbreviations and Terms", were performed at the sampling depths. The 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.

A relatively soft silty clay deposit was contacted at various depths at the site. A field vane shear test was performed in the soft clay stratum in one of the boreholes to determine its undrained shear strength.

Upon completion of borehole drilling, five (5) monitoring wells were installed at the selected borehole locations, to facilitate a hydrogeological assessment under a separate cover. The depth and details of the monitoring wells are shown on the corresponding Borehole Logs.

The ground elevation at each borehole location was determined with reference to a temporary bench mark, "Top of Catch Basin" located on the north side of Usman Road. It has a geodetic elevation of 90.50 m, as shown on the Site Plan of the project.

4.0 SUBSURFACE CONDITIONS

The site appeared to have been pregraded with stockpiles of earth fill in place. The investigation has disclosed that beneath a layer of earth fill, with surficial topsoil in isolated areas, the site is underlain by a silty clay deposit, overlying a glacial till stratum below 4.6 to 9.0 m from the prevailing ground surface, with occasional sand layers in the soil stratigraphy.

Detailed descriptions of the subsurface conditions are presented on the Borehole Logs, comprising Figures 1 to 10, inclusive. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing No. 2. The engineering properties of the disclosed soils are discussed herein.

4.1 **Topsoil** (Boreholes 2, 7 and 10)

Vegetation and topsoil was evident at the ground surface at the south portion of the property. The revealed topsoil ranges from 20 to 25 cm in thickness at the borehole locations. Thicker topsoil layers may occur beyond these boreholes, particularly at the low-lying area.

The existing stockpiles may also consist of topsoil or highly organic material. Test pits should be conducted to determine the soil composition in the stockpiles. The topsoil and stockpiles must be removed for site development.

4.2 **Earth Fill** (All Boreholes, except Boreholes 2, 7 and 8)

A layer of earth fill, extending to a depth of 1.0 to 2.4 m from the prevailing ground surface, was contacted below the ground surface at most of the boreholes. It consists of silty sand or sandy silt, with clay, gravel and occasional pockets of topsoil. The existing weed vegetation is anticipated to have impacted the earth fill at the ground surface. It must be stripped and removed prior to regrading the site for development.

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 earth fill, and do not indicate whether the topsoil beneath the earth fill was completely stripped. This should be further assessed by laboratory testing and/or test pits.

4.3 **Silty Clay** (All Boreholes)

The silty clay was contacted in the upper stratigraphy of boreholes. It is varved in structure, with occasional fine sand seams, showing a lacustrine deposit. Grain size analyses were performed on two representative samples; the results are plotted on Figure 11.

The obtained 'N' values in the silty clay stratum range from 0 (the weight of hammer) to 16, with a median of 5 blows per 30 cm of penetration, indicating soft to very stiff consistency.

Field vane shear test was performed in the soft clay in Borehole 9, the undrained shear strength is 24 kPa, having a sensitivity value of 2.

The Atterberg Limits of two silty clay samples and the natural water content values of all the clay samples were determined in the laboratory. The results are plotted on the Borehole Logs and summarized below:

Liquid Limit	29% and 33%
Plastic Limit	16% and 17%
Plasticity Index	13% and 16%
Natural Water Content	10% to 39% (median 24%)

The results show that the clay deposit is low to medium plasticity. The natural water content values varies in a wide range, probably affected by the presence of sand pockets or layers in the clay deposit.

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

- High frost susceptibility and soil-adfreezing potential.
- Low erodibility, except the laminated sand and silt layers in the deposit.
- Low permeability, with an estimated coefficient of permeability less than 10^{-7} cm/sec, a percolation rate above 60 min/cm, and runoff coefficients of:

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

- The shear strength is derived from consistency and augmented by the internal friction of the sand and silt. The overall shear strength 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.
- The soft clay may consolidate under excessive external loading.
- Any steep excavation into the soft clay may lead to base heaving due to overstressing.
- Moderately high corrosivity to buried metal, with an estimated electrical resistivity of 3000 ohm·cm.

4.4 **Glacial Till** (All Boreholes)

Glacial till deposit was contacted in the lower stratigraphy of boreholes, below 4.6 to 9.0 m from the prevailing ground surface. It is heterogeneous and amorphous in structure, consisting of a random mixture of particle sizes ranging from clay to gravel, with sand and silt being the dominant fraction. Tactile examinations of the soil samples indicated that the till is slightly cemented and displays some cohesion when remoulded. Hard resistance was encountered occasionally during augering and sampling, indicating the presence of cobbles and boulders in the deposit. Grain size analyses were performed on three representative samples; the results are plotted on Figures 12 and 13.

The obtained 'N' values range from 21 to over 100 blows per 30 cm of penetration, indicating the till deposit is compact to very dense in relative density. The natural water content values of the till samples range from 6% to 15%, with a median of 9%, indicating damp to moist conditions.

The following engineering properties of the till deposit are deduced:

- Moderately high frost susceptibility and soil-adfreezing potential.
- Moderately low water erodibility.
- Low permeability, with an estimated coefficient of permeability of 10^{-6} cm/sec, a percolation rate of 50 min/cm, and runoff coefficients of:

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

- A frictional soil, the shear strength is primarily derived from internal friction, and is augmented by cementation. The soil strength is density dependent.
- The till will generally be stable in steep cuts; however, with prolonged exposure, localized sheet collapse will likely occur.
- Moderate corrosivity to buried metal, with an estimated electrical resistivity of 4500 ohm·cm.

4.5 **Sand** (Boreholes 2, 6, 7 and 8)

The sand deposit was contacted near the ground surface of Boreholes 2, 6, 7 and 8, and at a depth of 11.6 m, below the glacial till at Borehole 7 location. It is fine grained with silt.

The obtained 'N' values range from 2 to over 52 blows per 30 cm of penetration. The deposit near the ground surface is generally loose, probably due to weathering or ground disturbance from the previous earth work at the site. The lower sand deposit, however, is very dense in relative density.

The water content values of the sand samples are plotted on the borehole logs. The values range from 4% to 18%, with a median of 14%, indicating damp to wet conditions.

The following engineering properties of the sand deposit are deduced:

- Moderate to high frost susceptibility.
- High water erodibility; it is susceptible to migration through small openings under seepage pressure.
- Pervious, with an estimated coefficient of permeability of 10^{-3} to 10^{-4} cm/sec, a percolation rate of 10 to 15 minutes/cm and runoff coefficients of:

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

- The shear strength is density dependent. Due to its dilatancy, the strength of the wet sand is susceptible to impact disturbance.
- In excavation, the sand will slough and run slowly with seepage bleeding from the cut face. It will boil with a piezometric head of about 0.3 m.
- Low corrosivity to buried metal, having an estimated electrical resistivity of 6500 ohm·cm.

5.0 GROUNDWATER CONDITIONS

Free groundwater was recorded in the boreholes upon completion of drilling. The groundwater readings in the monitoring wells were also recorded on November 14, 2019. The data are summarized in Table 1.

Table 1 - Groundwater Level in Open Boreholes and Monitoring Wells

Borehole No.	Ground Elevation (m)	Groundwater/Cave-in* Level On Completion		Water Level in Monitoring Well on November 14, 2019	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
1 (MW)	91.0	Dry	-	5.3	85.7
2	90.0	Dry	-	No Well	
3	91.0	8.5	82.5	No Well	
4 (MW)	90.3	Dry	-	6.6	83.7
5	90.9	8.8/10.4	82.1/80.5*	No Well	
6	91.8	7.0	84.8	No Well	
7	89.5	9.5/11.3*	80.0/78.2*	No Well	
8 (MW)	88.7	Dry	-	5.9	82.8
9 (MW)	87.3	Dry	-	4.7	82.6
10 (MW)	85.9	8.5	77.4	3.1	82.8

The groundwater recorded in the boreholes, between 8.5 m and 9.5 m from grade, might represent perched water in the sand seams or layers, except in Borehole 7, where saturated sand deposit was contacted below the till stratum, at a depth of 11.6 m.

Stabilized groundwater was recorded in the wells between 3.1 m and 6.6 m from grade, or between El. 85.7 m and El. 82.6 m. It is subject to seasonal fluctuation.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation has disclosed that beneath a layer of earth fill, with surficial topsoil in isolated areas, the site is underlain by soft to very stiff silty clay deposit, overlying a compact to very dense glacial till stratum below 4.6 to 9.0 m from the prevailing ground surface, with occasional sand layers in the soil stratigraphy.

The stabilized groundwater was recorded in the monitoring wells, between El. 85.7 m and El. 82.6 m. It is subject to seasonal fluctuation.

The proposed development includes a residential building (Block A) of 6 and 20 storeys, connected by a 4-storey podium at the west portion. The east portion will consist of three structures (Blocks B, C and D) of two stacked townhouse blocks and one street townhouse block. These buildings will be constructed on an adjoined underground parking, with a lower parking level at the west portion.

According to the Architectural Plan, the design elevation of the underground parking is:

- El. 80.78 m for the lower parking deck of Block A;
- El. 83.78 m for the adjoined parking deck of Blocks B, C and D.

The geotechnical findings warranting special consideration for the proposed development are presented below:

1. The founding elevation of Block A will extend to El. 80.0 to 79.5 m for the lower parking deck, where dense till is anticipated. Conventional footings and raft foundation can be constructed on the dense till at the design level.
2. The founding elevation of Blocks B, C and D will extend to El. 83.0 to 82.5 m where silty clay is anticipated. The clay is generally soft, with limited capacity for foundation design.
3. Where the space is not sufficient for a safe backing slope, the excavation should be supported by a braced shoring system.
4. Due to the low permeability of the glacial till and silty clay, localized dewatering from sumps will be sufficient for excavation and construction.

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

6.1 Foundations

The proposed development includes a residential building (Block A) of 6 and 20 storeys, connected by a 4-storey podium at the west portion. The east portion will consist of three structures (Blocks B, C and D) of two stacked townhouse blocks and one street townhouse block. These buildings will be constructed on an adjoined underground parking, with a lower parking level at the west portion. According to the Architectural Plan, the design elevation of the underground parking is:

- El. 80.78 m for the lower parking deck of Block A;
- El. 83.78 m for the adjoined parking deck of Blocks B, C and D.

Block A

The foundation for Block A will extend to El. 80 to 79.5 m where dense till is anticipated. The native till deposit is capable to support the proposed structure on conventional footings or raft foundation.

For the design of conventional spread and strip footings in dense till deposit, the recommended soil bearing pressures are provided:

- Maximum Soil Bearing Pressure at Serviceability Limit State (SLS) = 800 kPa
- Ultimate Limit State (ULS) Bearing Capacity = 1200 kPa

The maximum footing width should be limited within 6 m, having the estimated total and differential settlements of footings within 25 mm and 20 mm, respectively.

One must be aware that for conventional footing construction, the underground structure must be provided with perimeter drainage connecting into the sump pit where water can be removed into the municipal sewer system. If the Municipality does not allow the permanent drainage of subsurface water into the sewer system, a separate storage cistern should be provided or, otherwise, the underground structure must be waterproofed and designed to resist the full hydrostatic pressure. A tank structure with waterproofed raft foundation is appropriate to resist the hydrostatic pressure and buoyance uplift. The recommended soil bearing pressures for raft foundation or footings exceeding 6 m in width are provided in the dense till deposit:

- Maximum Soil Bearing Pressure at Serviceability Limit State (SLS) = 600 kPa
- Ultimate Limit State (ULS) Bearing Capacity = 1000 kPa

The total and differential settlements are estimated within 25 mm and 15 mm, respectively. A Modulus of Subgrade Reaction of 60 MPa/m can be used for the design of raft foundation.

Blocks B, C and D

The foundation will extend to El. 83.0 to 82.5 m where silty clay is anticipated. The clay is generally soft. The recommended soil bearing pressures for the design of footings or raft foundation on the clay stratum are provided:

- Net Allowable Soil Bearing Pressure (SLS) = 75 kPa
- Factored Ultimate Soil Bearing Pressure (ULS) = 120 kPa

The total and differential settlements of conventional footings or raft foundation are estimated to be 25 mm and 20 mm, respectively. A Modulus of Subgrade Reaction of 7.5 MPa/m can be used for the design of raft foundation.

The underground structure must also be provided with perimeter drainage connecting into the sump pit where water can be removed into the municipal sewer system or a storage cistern. Otherwise, the underground structure must be waterproofed and designed to resist the full hydrostatic pressure.

As an alternative, the townhouse blocks can be supported on drilled concrete piers (caissons) extending into the very dense till stratum. The recommended bearing pressures of caissons, extending at least 1 m into the till deposit, are as follows:

- Maximum Allowable Axial End Bearing Pressure (SLS) = 900 kPa
- Factored Ultimate Axial Bearing Pressure (ULS) = 1800 kPa

In order to facilitate inspections and base cleaning, the caissons should be at least 80 cm in diameter and must be temporarily lined for safety and to prevent cave-in. The ratio of the embedded depth to the diameter of caisson should be at least 2:1. The centre-to centre spacing between the caissons must be at least three times the diameter of the caisson base.

The total and differential settlements of caissons are estimated at 20 mm and 15 mm, respectively.

Joints and Connections

Slip-joints should be provided at the connections of structures having different structural configurations and anticipated settlement. This is to allow for abrupt differential settlement at the interface without imposing structural distress on the structures constructed with different foundations.

Construction of Foundation

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

A mud slab of lean mix concrete, 8 to 10 cm in thickness, will be required to provide a working platform for the workers to install the reinforcement in the raft foundation. Similarly, a mud slab will be required in any footing excavation where groundwater seepage is evident, after the subgrade soil is inspected and approved by the geotechnical engineer. This will prevent construction disturbance and costly rectification.

Foundations exposed to weathering, or in unheated areas, should have at least 1.2 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 0.6 m for the perimeter walls and 0.9 m for the interior walls and columns, except in the area in close proximity to ventilation shafts and the door entrances.

Site Classification for Seismic Design

Block A, having the building foundation extending into the dense till stratum, should be designed to resist an earthquake force using Site Classification 'C' (very dense soil).

Blocks B, C and D should be designed to resist an earthquake force using Site Classification 'E' (soft soil) for foundation founded on the clay deposit. If the building blocks are supported on caissons extending into the dense till stratum, they should be designed to resist an earthquake force using Site Classification 'C' (very dense soil).

6.2 Underground Garage and Slab-On-Grade

The design elevation of the underground parking is:

- El. 80.78 m for the lower parking deck of Block A;
- El. 83.78 m for the adjoined parking deck of Blocks B, C and D.

The design floor elevation of the underground parking is below the stabilized groundwater recorded in the monitoring wells, between El. 85.7 m and El. 82.6 m. It is necessary to control the groundwater by permanent drainage, including the perimeter drainage and subfloor drainage systems. The elevator pit, which normally extends below the floor level, should be designed as a submerged 'tank' structure with waterproofed pit walls and pit floor.

In conventional design, the perimeter walls of underground structures should be dampproofed and provided with a perimeter subdrain encased in a fabric filter at the wall base. At the shoring location, prefabricated drainage board, such as Miradrain 6000 or equivalent, must be provided between the shoring wall and the cast-in-place foundation wall (Drawing No. 3). The subfloor subdrain system, consisting of 100-mm filter-sleeved weepers, should also be installed in a grid pattern, not more than 6 m on centres, at a depth of 0.6 m from the slab-on-grade. A vapour barrier should also be provided at the top of the floor bedding, above the subfloor drains, to prevent the emission of water vapours. Details of the underfloor subdrains are presented in Drawing No. 4. The perimeter and the subfloor drainage systems must be connected separately to the sump-pit where water can be removed.

The subgrade for conventional slab-on-grade construction should consist of sound natural soils or properly compacted inorganic earth fill. The slab should be constructed on a granular bedding of 20 cm thick, consisting of 20-mm Crusher-Run Limestone, or equivalent.

The perimeter walls 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.

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

At the garage entrances, the subgrade should be properly insulated, or the subgrade material should be replaced with 1.2 m of non-frost-susceptible granular material and should be provided with subdrains. This will minimize frost action in this area where vertical ground movement cannot be tolerated. The floor at the entrance and in areas of close proximity to air shafts should be insulated, and the insulation should extend 1.5 m internally. This measure is to prevent frost action induced by cold drafts. The exterior grade should slope away from the building to prevent ponding of water in the areas adjacent to the building.

The exterior grading should slope such that surface runoff is directed away from the building or structure. This is to prevent ponding adjacent to the underground garage.

6.3 **Underground Services**

The subgrade for underground services should consist of properly compacted inorganic earth fill or sound natural soils. A Class ‘B’ bedding, consisting of compacted 20-mm Crusher-Run Limestone or equivalent, is recommended for the design of the underground services construction. The pipe joints should be leak-proof or wrapped with a waterproof membrane.

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

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

All metal fittings for the underground services should be protected against soil corrosion. The in situ soils have moderately high to low corrosivity to buried metal. In determining the mode of protection, an electrical resistivity of 3000 ohm·cm should be used. This, however, should be confirmed by testing the soil along the water main alignment at the time of services construction.

6.4 Trench Backfilling

The backfill in service trenches or beside foundation walls should be compacted to at least 95% of its maximum Standard Proctor Dry Density (SPDD). In the zone within 1.0 m below the pavement or floor subgrade, the material should be compacted with the water content 2% to 3% drier than the optimum, compacted to 98% of the respective maximum SPDD.

On site inorganic soils are suitable for use as trench backfill; where the in situ soils are too wet, they must be aerated by spreading them thinly on the ground during the dry, warm weather before compaction.

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

6.5 Interlocking Stone Pavement, Sidewalk and Landscaping

Interlocking stone pavement, concrete sidewalk and landscaping structures in areas which are sensitive to frost-induced ground movement, such as in front of building entrances, must be constructed on a free-draining, non-frost-susceptible granular material such as Granular 'B'. This material must extend to at least 0.3 to 1.2 m below the slab or pavement surface, depending on the degree of tolerance of ground movement, and be provided with positive drainage, such as weeper subdrains connected to manholes or catch basins. Alternatively, they should be properly insulated with 50-mm Styrofoam, or equivalent.

The grading around structures must be such that it directs runoff away from the structures.

6.6 Pavement Design

The proposed pavement for the development will be built on a structural slab over the underground garage rooftop. A sufficient granular base and adequate drainage must be provided to prevent frost damage to the pavement. A waterproof membrane must be placed above the structural slab exposed to weathering to prevent water leakage, as well as to protect the reinforcing steel bars against brine corrosion.

The recommended pavement structure to be placed on the roof of the underground garage is presented in Table 2.

Table 2 - Pavement Design (Roof of Underground Garage)

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

For the on-grade access driveway and fire route between the road and the building, the recommended pavement design is presented in Table 3.

Table 3 - Pavement Design (On-Grade Access Driveway)

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	50	HL-8
Granular Base	150	Granular ‘A’ or equivalent
Granular Sub-base	300	Granular ‘B’ or equivalent

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

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

In the zone within 1.0 m below the pavement subgrade, the backfill should be compacted to at least 98% of its maximum Standard Proctor dry density, with the water content 2% to 3% drier than the optimum. In the lower zone, a 95% or + Standard Proctor compaction is considered adequate.

Along the perimeter at the lower spots where surface runoff may drain onto the pavement, a swale or an intercept subdrain system should be installed to prevent the infiltrating surface water from seeping into the granular bases (since this may inflict frost damage on the flexible pavement). The subdrains should consist of filter-wrapped weepers, and they should be connected to the catch basins or 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 4.

Table 4 - Soil Parameters

Unit Weight and Bulk Factor	**Unit Weight γ (kN/m^3)**		**Estimated Bulk Factor**	
	Bulk	**Submerged**	**Loose**	**Compacted**
Earth Fill/Silty Clay	21.0	11.0	1.20	0.95
Glacial Till	22.5	12.5	1.30	1.03
Sand	21.5	11.5	1.25	1.00
Lateral Earth Pressure Coefficients		**Active K_a**	**At Rest K_o**	**Passive K_p**
Compacted Earth Fill/Silty Clay		0.40	0.55	2.50
Glacial Till/Sand		0.30	0.45	3.30
Coefficients of Friction				
Between Concrete and Granular Base				0.50
Between Concrete and Sound Natural Soils				0.35

6.8 Excavation

Excavation should be carried out in accordance with Ontario Regulation 213/91. The types of soils are classified in Table 5.

Table 5 - Classification of Soils for Excavation

Material	Type
Glacial Till	2
Earth Fill, stiff to very stiff Silty Clay and drained Sand	3
Saturated Soils and very soft to firm Silty Clay	4

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

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

During excavation and construction, temporary dewatering by conventional pumping from sumps will be required if seepage of groundwater is encountered. In order to optimize the effect of the dewatering system, we recommend that the installation of soldier piles for the shoring system should be carefully monitored to record the contact elevation of the soils below saturation and the static water levels. This information should be reviewed by the dewatering contractor and Soil Engineers Ltd., in order to determine the necessary extent of the dewatering system on site.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of Brock Road Duffins Forest Inc. and for review by its designated agents, consultants, financial institutions, 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 judgment of Kelvin Hung, P.Eng., 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, and/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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Bennett Sun, P.Eng.
KH/BS:dd

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.: 1909-S140

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed Residential Development

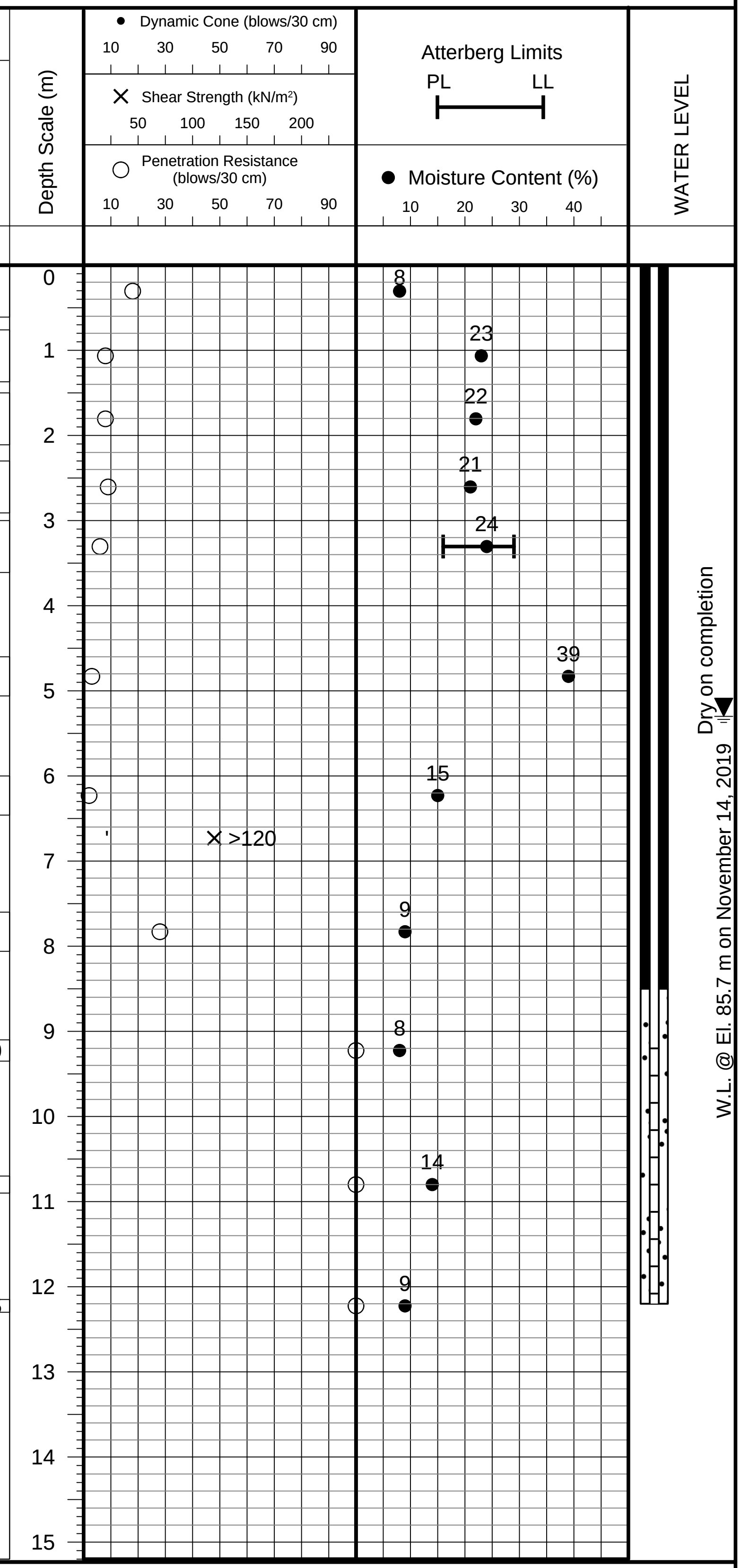
METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2055 Brock Road, City of Pickering

DRILLING DATE: October 15, 2019

El. (m) / Depth (m)	SOIL DESCRIPTION	Sample Number	Type	N-Value
91.0	Ground Surface			
0.0	Dark brown **EARTH FILL** silty sand to sandy silt occ. topsoil pockets	1	DO	18
		2	DO	8
89.3 / 1.7	Soft to stiff **SILTY CLAY** — brown / grey; a trace of sand, occ. sand seams and layers	3	DO	8
		4	DO	9
		5	DO	6
		6	DO	3
		7	DO	2
84.0 / 7.0	Grey, compact to very dense **GLACIAL TILL** sandy silt to silty sand till, traces to some clay and gravel, occ. cobbles and boulders	8	DO	28
		9	DO	50/10
		10	DO	50/5
		11	DO	55/15
78.7 / 12.3	**END OF BOREHOLE**			

Installed 50 mm Ø monitoring well to 12.2 m
completed with 3.0 m screen
Sand backfill from 8.5 m to 12.2 m
Bentonite seal from 0.0 m to 8.5 m
Provided with a steel casing.



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LOG OF BOREHOLE NO.: 2

JOB NO.: 1909-S140

FIGURE NO.: 2

PROJECT DESCRIPTION: Proposed Residential Development

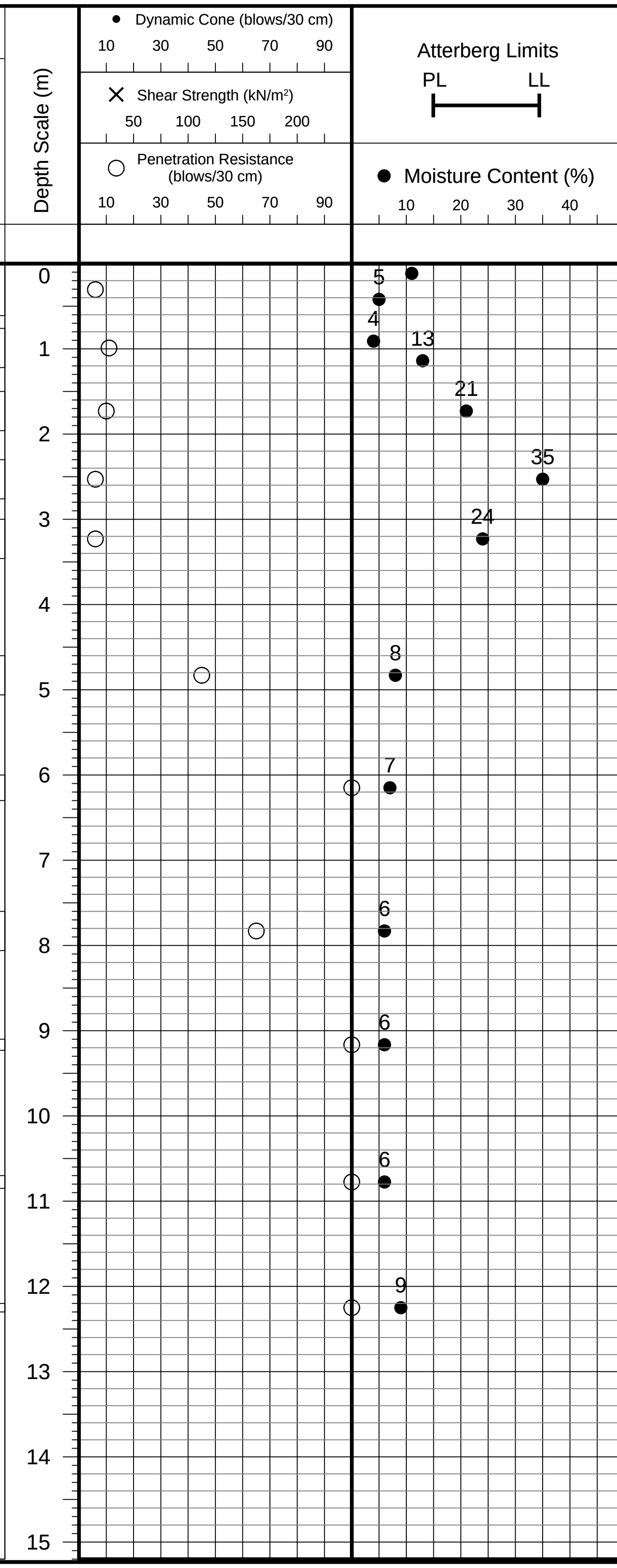
METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2055 Brock Road, City of Pickering

DRILLING DATE: October 15 and 17, 2019

El. (m) / Depth (m)	SOIL DESCRIPTION
90.0	Ground Surface
0.0	**23 cm TOPSOIL**
	Brown, compact, weathered **SAND** fine-grained, some silt to silty
88.5 / 1.5	Brown, firm to stiff **SILTY CLAY** a trace of sand occ. sand seams and layers
85.4 / 4.6	Grey, dense to very dense **GLACIAL TILL** sandy silt till to silty sand till traces to some clay and gravel occ. cobbles and boulders
77.7 / 12.3	**END OF BOREHOLE**

SAMPLES Number	Type	N-Value
1A		
1B	DO	6
2A		
2B	DO	11
3	DO	10
4	DO	6
5	DO	6
6	DO	45
7	DO	50/15
8	DO	65
9	DO	50/13
10	DO	60/15
11	DO	50/10



WATER LEVEL

Dry on completion

JOB NO.: 1909-S140

LOG OF BOREHOLE NO.: 3

FIGURE NO.: 3

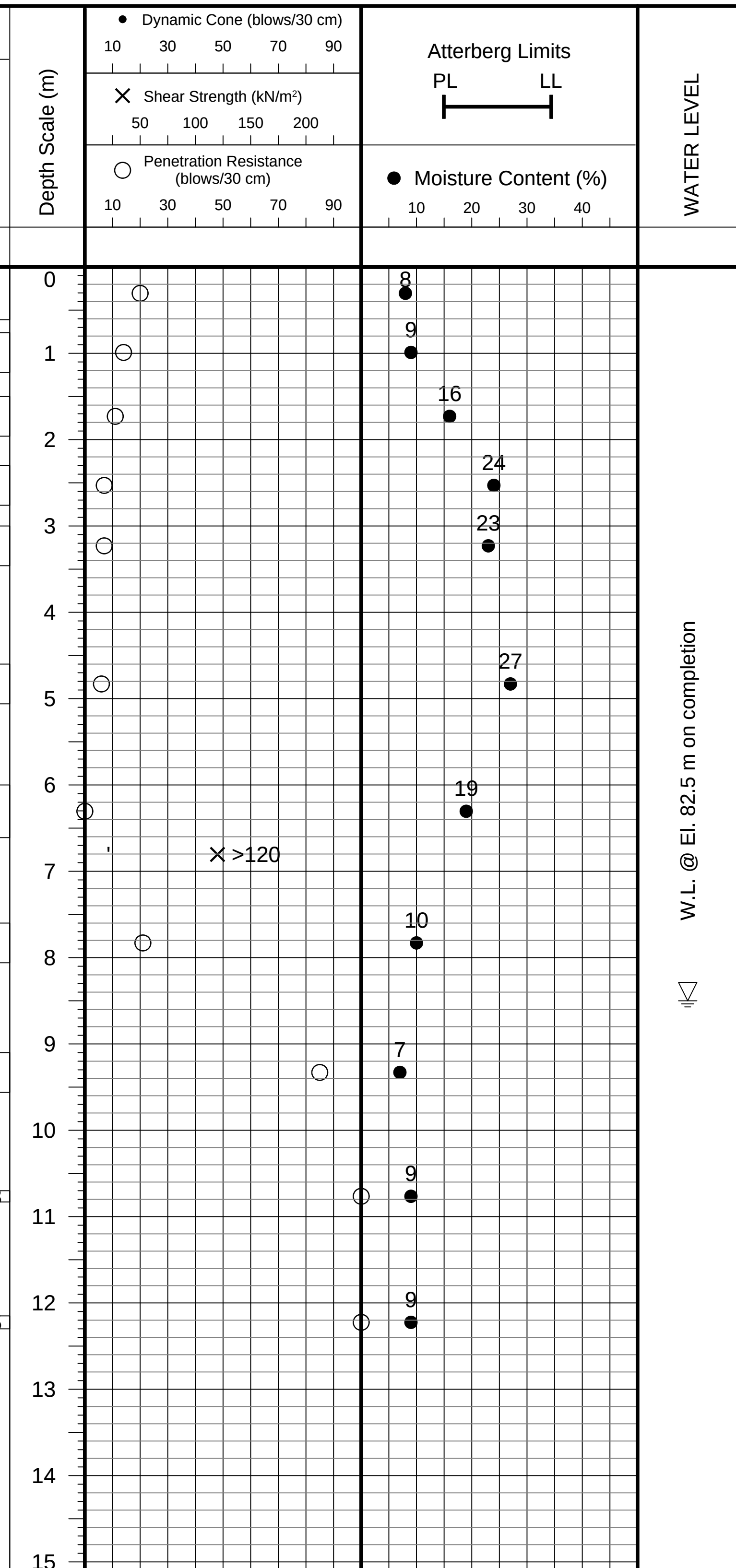
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2055 Brock Road, City of Pickering

DRILLING DATE: October 17, 2019

El. (m) / Depth (m)	SOIL DESCRIPTION	Sample Number	Type	N-Value
91.0	Ground Surface			
0.0	Brown to grey **EARTH FILL** silty sand to sandy silt	1	DO	20
		2	DO	14
		3	DO	11
88.6 / 2.4	Soft to stiff **SILTY CLAY** a trace of sand occ. sand seams and layers (brown / grey)	4	DO	7
		5	DO	7
		6	DO	6
		7	DO	0
84.0 / 7.0	Grey, compact to very dense **GLACIAL TILL** sandy silt till to silty sand till traces to some clay and gravel occ. cobbles and boulders	8	DO	21
		9	DO	85
		10	DO	50/13
78.7 / 12.3	**END OF BOREHOLE**	11	DO	60/15



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JOB NO.: 1909-S140

LOG OF BOREHOLE NO.: 4

FIGURE NO.: 4

PROJECT DESCRIPTION: Proposed Residential Development

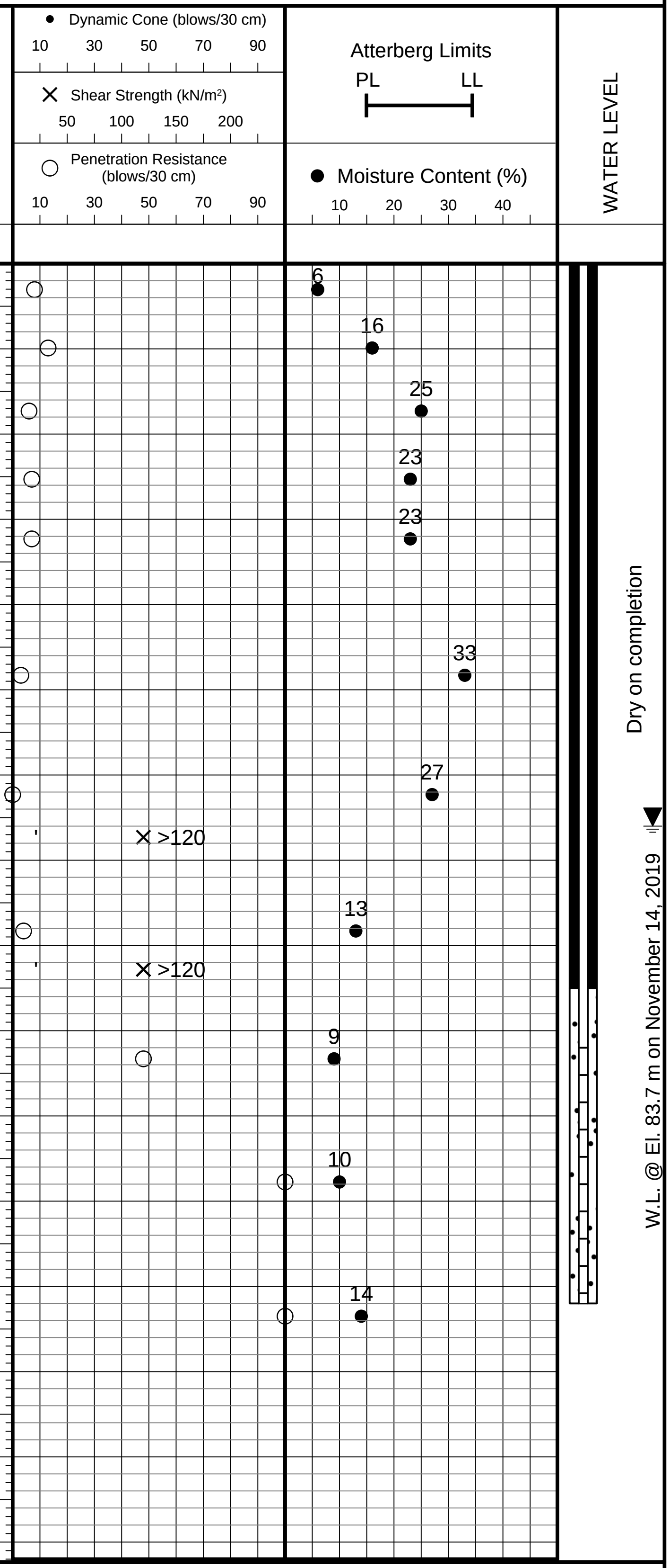
METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2055 Brock Road, City of Pickering

DRILLING DATE: October 15, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	Sample Number	Type	N-Value	Depth Scale (m)
90.3	Ground Surface				
0.0	Brown **EARTH FILL** silty sand, occ. topsoil pockets	1	DO	8	0
		2	DO	13	1
88.6 / 1.7	Soft to firm **SILTY CLAY** (brown / grey at approx. 4.0 m) a trace of sand, occ. sand seams and layers	3	DO	6	
		4	DO	7	2
		5	DO	7	3
		6	DO	3	5
		7	DO	0	6
		8	DO	4	8
81.3 / 9.0	Grey, dense to very dense **GLACIAL TILL** sandy silt to silty sand till, traces to some clay and gravel, occ. cobbles and boulders	9	DO	48	9
		10	DO	50/15	11
		11	DO	50/15	12
77.8 / 12.5	**END OF BOREHOLE**				

Installed 50 mm Ø monitoring well to 12.2 m
completed with 3.0 m screen
Sand backfill from 8.5 m to 12.2 m
Bentonite seal from 0.0 m to 8.5 m
Provided with a steel casing.



JOB NO.: 1909-S140

LOG OF BOREHOLE NO.: 5

FIGURE NO.: 5

PROJECT DESCRIPTION: Proposed Residential Development

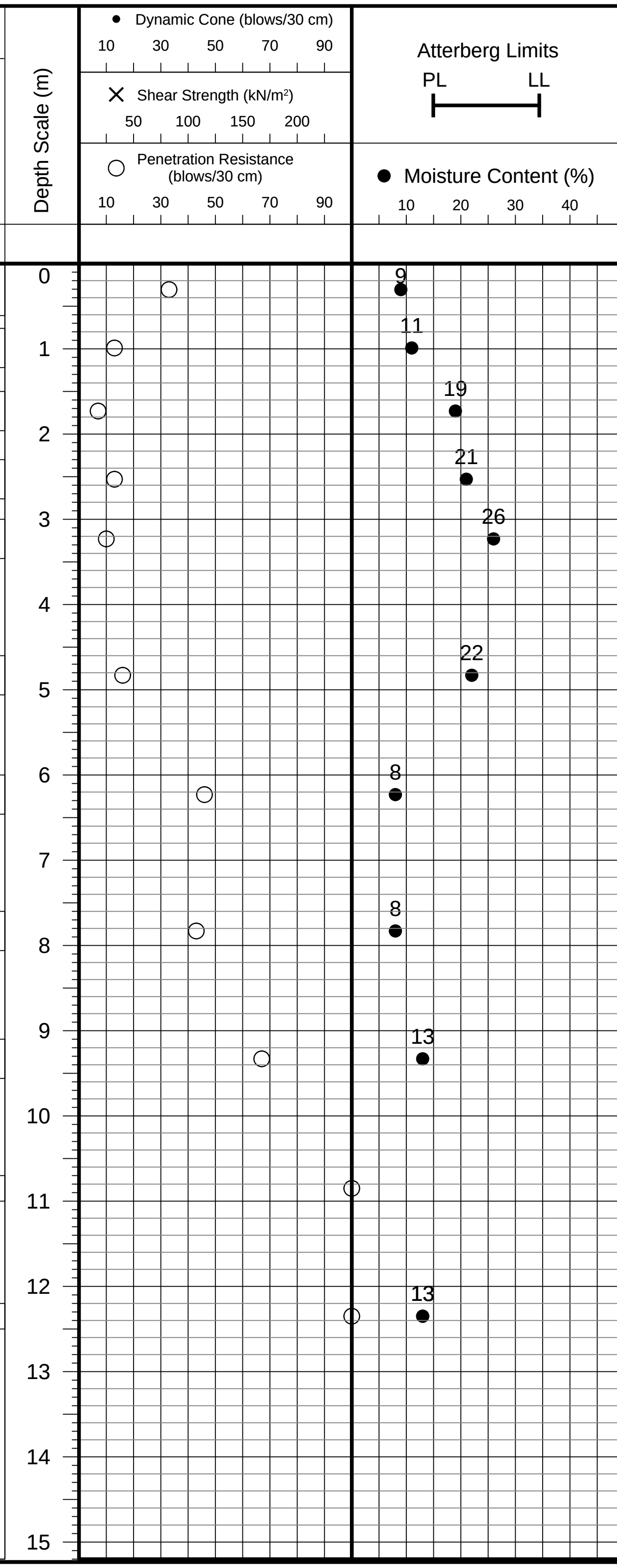
METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2055 Brock Road, City of Pickering

DRILLING DATE: October 11, 2019

El. (m) / Depth (m)	SOIL DESCRIPTION
90.9	Ground Surface
0.0	Dark brown **EARTH FILL** silty clay with rootlets and topsoil pockets
88.5 / 2.4	Brown, stiff to very stiff **SILTY CLAY** a trace of sand occ. sand seams and layers
84.9 / 6.0	Grey, dense to very dense **GLACIAL TILL** sandy silt till to silty sand till traces to some clay and gravel occ. cobbles and boulders
78.4 / 12.5	**END OF BOREHOLE**

SAMPLES Number	Type	N-Value	Moisture Content (%)
1	DO	33	9
2	DO	13	11
3	DO	7	19
4	DO	13	21
5	DO	10	26
6	DO	16	22
7	DO	46	8
8	DO	43	8
9	DO	67	13
10	DO	60/15	
11	DO	65/15	13



WATER LEVEL

W.L. @ El. 82.1 m on completion

Cave-in @ El. 80.5 m on completion

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LOG OF BOREHOLE NO.: 6

JOB NO.: 1909-S140

FIGURE NO.: 6

PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2055 Brock Road, City of Pickering

DRILLING DATE: October 17, 2019

El. (m) / Depth (m)	SOIL DESCRIPTION
91.8	Ground Surface
0.0	Brown **EARTH FILL** silty clay with gravel occ. sand layers and topsoil pockets
90.3 / 1.5	Brown, loose, weathered **SAND** fine-grained with clay layer
89.5 / 2.3	Stiff to very soft **SILTY CLAY** a trace of sand occ. sand seams and layers
84.2 / 7.6	Grey, very dense **GLACIAL TILL** sandy silt till to silty sand till traces to some clay and gravel occ. cobbles and boulders
79.8 / 12.0	**END OF BOREHOLE** Refusal to augering, inferred boulder

SAMPLES Number	Type	N-Value	Moisture Content (%)
1	DO	13	11
2	DO	12	11
3	DO	6	18
4	DO	11	21
5	DO	12	22
6	DO	2	35
7	DO	1	24
8	DO	70	8
9	DO	52	8
10	DO	50/8	7

Depth Scale (m): 0 – 15

● Dynamic Cone (blows/30 cm): 10, 30, 50, 70, 90

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

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

Atterberg Limits: PL, LL

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

WATER LEVEL: W.L. @ El. 84.8 m on completion

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JOB NO.: 1909-S140

LOG OF BOREHOLE NO.: 7

FIGURE NO.: 7

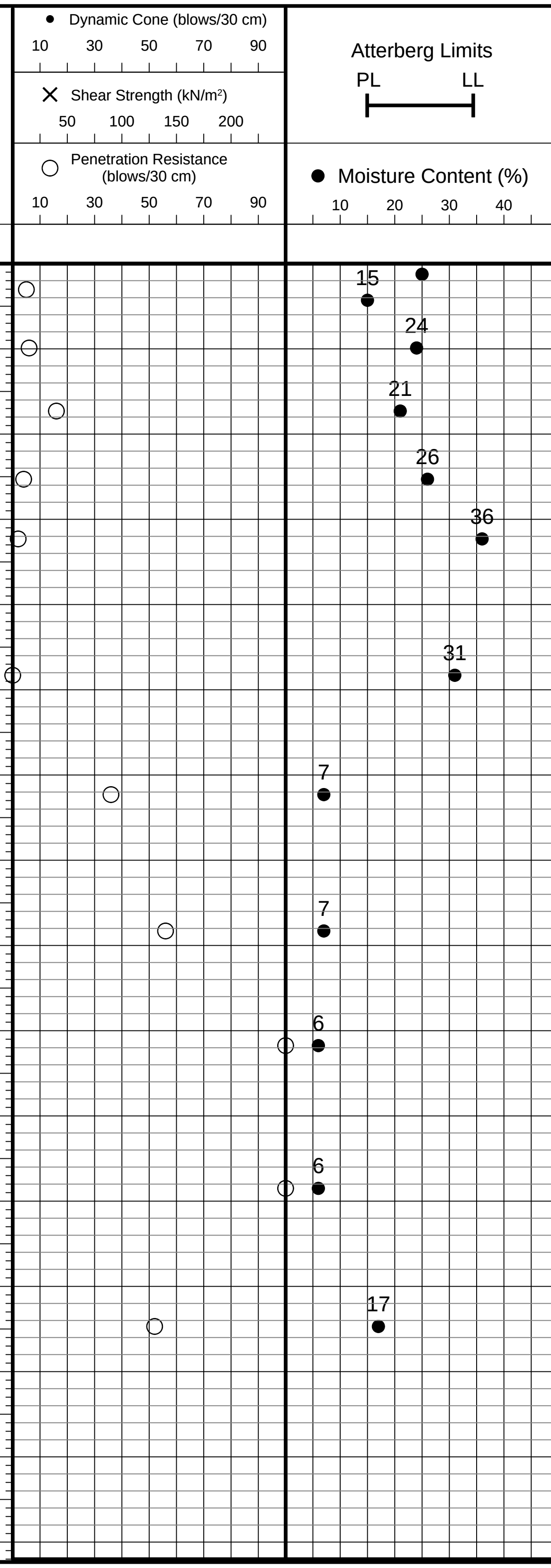
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2055 Brock Road, City of Pickering

DRILLING DATE: October 21, 2019

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)
89.5	Ground Surface				
0.0	**25 cm TOPSOIL**	1A	DO	5	0
	Brown, loose, weathered **SAND** fine-grained, some silt to silty	1B			
88.6 / 0.9	Brown, stiff to very soft	2	DO	6	1
		3	DO	16	2
	SILTY CLAY	4	DO	4	
	brown / grey	5	DO	2	3
	a trace of sand occ. sand seams and layers				4
		6	DO	0	5
83.5 / 6.0	Grey, dense to very dense	7	DO	36	6
					7
	GLACIAL TILL	8	DO	56	8
		9	DO	55/15	9
	sandy silt till to silty sand till traces to some clay and gravel occ. cobbles and boulders				10
		10	DO	50/15	11
77.9 / 11.6	Grey, very dense **SAND** fine-grained occ. silt seams	11	DO	52	12
76.8 / 12.7	**END OF BOREHOLE**				13
					14
					15



WATER LEVEL

W.L. @ El. 80.0 m on completion

Cave-in @ El. 78.2 m on completion

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JOB NO.: 1909-S140

LOG OF BOREHOLE NO.: 8

FIGURE NO.: 8

PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2055 Brock Road, City of Pickering

DRILLING DATE: October 21, 2019

Legend: ● Dynamic Cone (blows/30 cm) 10 30 50 70 90; × Shear Strength (kN/m²) 50 100 150 200; ○ Penetration Resistance (blows/30 cm) 10 30 50 70 90; Atterberg Limits PL LL; ● Moisture Content (%) 10 20 30 40

El. (m) / Depth (m)	SOIL DESCRIPTION	Sample Number	Type	N-Value	Moisture Content (%)
88.7	Ground Surface				
0.0	Brown, very loose, weathered **SAND** fine-grained, some silt to silty	1	DO	2	8
		2A	DO	3	15
87.7 / 1.0	Brown, stiff to very soft	2B			23
		3	DO	8	23
		4	DO	6	25
	SILTY CLAY (brown / grey)	5	DO	3	24
	a trace of sand, occ. sand seams and layers	6	DO	0	27
		7	DO	1	32
81.1 / 7.6	Grey, very dense	8	DO	50/8	10
	GLACIAL TILL	9	DO	60/15	9
	sandy silt till to silty sand till, traces to some clay and gravel, occ. cobbles, boulders and shale fragments	10	DO	50/8	10
76.4 / 12.3		11	DO	55/15	8
	END OF BOREHOLE				

Installed 50 mm Ø monitoring well to 11.7 m
completed with 3.0 m screen
Sand backfill from 8.0 m to 11.7 m
Bentonite seal from 0.0 m to 8.0 m
Provided with a steel casing.

WATER LEVEL: Dry on completion; W.L. @ El. 82.8 m on November 14, 2019

Depth Scale (m): 0–15

Soil Engineers Ltd.

JOB NO.: 1909-S140

LOG OF BOREHOLE NO.: 9

FIGURE NO.: 9

PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2055 Brock Road, City of Pickering

DRILLING DATE: October 18, 2019

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Moisture Content (%)
87.3	Ground Surface				
0.0	Brown **EARTH FILL** silty sand, some clay	1	DO	4	12
86.3 / 1.0	Brown, very soft to stiff	2	DO	9	24
		3	DO	5	22
	brown / grey	4	DO	4	24
		5	DO	0	28
	SILTY CLAY				
		6	DO	0	35
	traces of sand and gravel occ. sand seams and layers	7	DO	0	31
		8	DO	8	10
78.3 / 9.0	Grey, very dense	9A / 9B	DO	52	9 / 15
	sand layer				
	GLACIAL TILL				
	sandy silt till to silty sand till traces to some clay and gravel occ. cobbles, boulders and shale fragments	10	DO	50/8	11
75.0 / 12.3	**END OF BOREHOLE**	11	DO	50/10	9

Installed 50 mm Ø monitoring well to 12.2 m
completed with 3.0 m screen
Sand backfill from 8.5 m to 12.2 m
Bentonite seal from 0.0 m to 8.5 m
Provided with a steel casing.

Legend: ● Dynamic Cone (blows/30 cm) 10 30 50 70 90; × Shear Strength (kN/m²) 50 100 150 200; ○ Penetration Resistance (blows/30 cm) 10 30 50 70 90; Atterberg Limits PL LL; ● Moisture Content (%) 10 20 30 40; Depth Scale (m) 0–15; WATER LEVEL

×ST 2.0

Dry on completion

W.L. @ El. 82.6 m on November 14, 2019

Soil Engineers Ltd.

JOB NO.: 1909-S140

LOG OF BOREHOLE NO.: 10

FIGURE NO.: 10

PROJECT DESCRIPTION: Proposed Residential Development

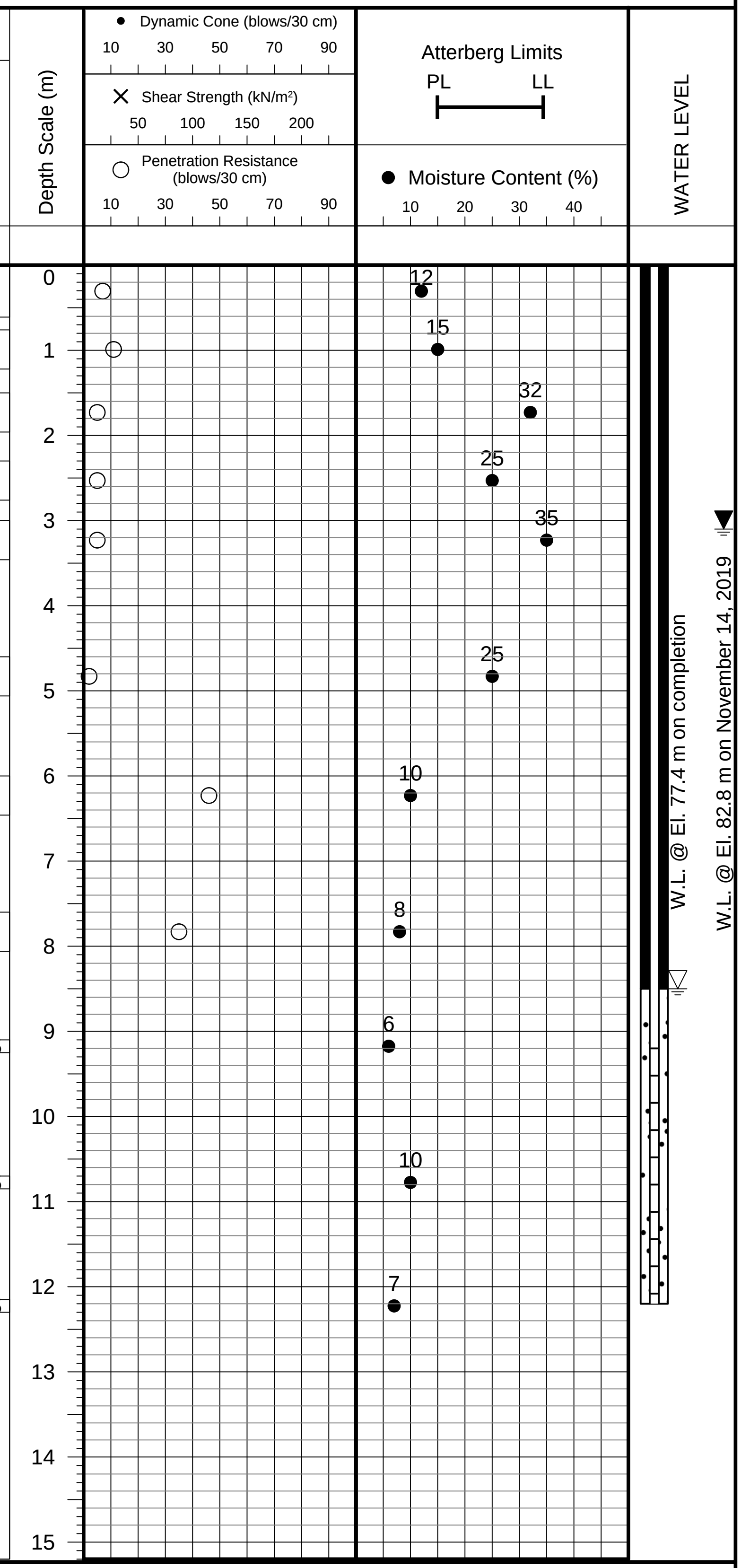
METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2055 Brock Road, City of Pickering

DRILLING DATE: October 18, 2019

El. (m) / Depth (m)	SOIL DESCRIPTION	Sample Number	Type	N-Value	Moisture Content (%)
85.9	Ground Surface				
0.0	**20 cm TOPSOIL**				
	Brown **EARTH FILL** silty clay, topsoil pockets	1	DO	7	12
		2	DO	11	15
84.4 / 1.5	Brown, firm to very soft **SILTY CLAY** traces of sand and gravel occ. sand seams and layers	3	DO	5	32
		4	DO	5	25
		5	DO	5	35
		6	DO	2	25
79.9 / 6.0	Grey, dense to very dense **GLACIAL TILL** sandy silt till to silty sand till traces to some clay and gravel occ. cobbles, boulders and shale fragments	7	DO	46	10
		8	DO	35	8
		9	DO	50/15	6
		10	DO	50/15	10
		11	DO	50/15	7
73.6 / 12.3	**END OF BOREHOLE**				

Installed 50 mm Ø monitoring well to 12.2 m completed with 3.0 m screen
Sand backfill from 8.5 m to 12.2 m
Bentonite seal from 0.0 m to 8.5 m
Provided with a steel casing.



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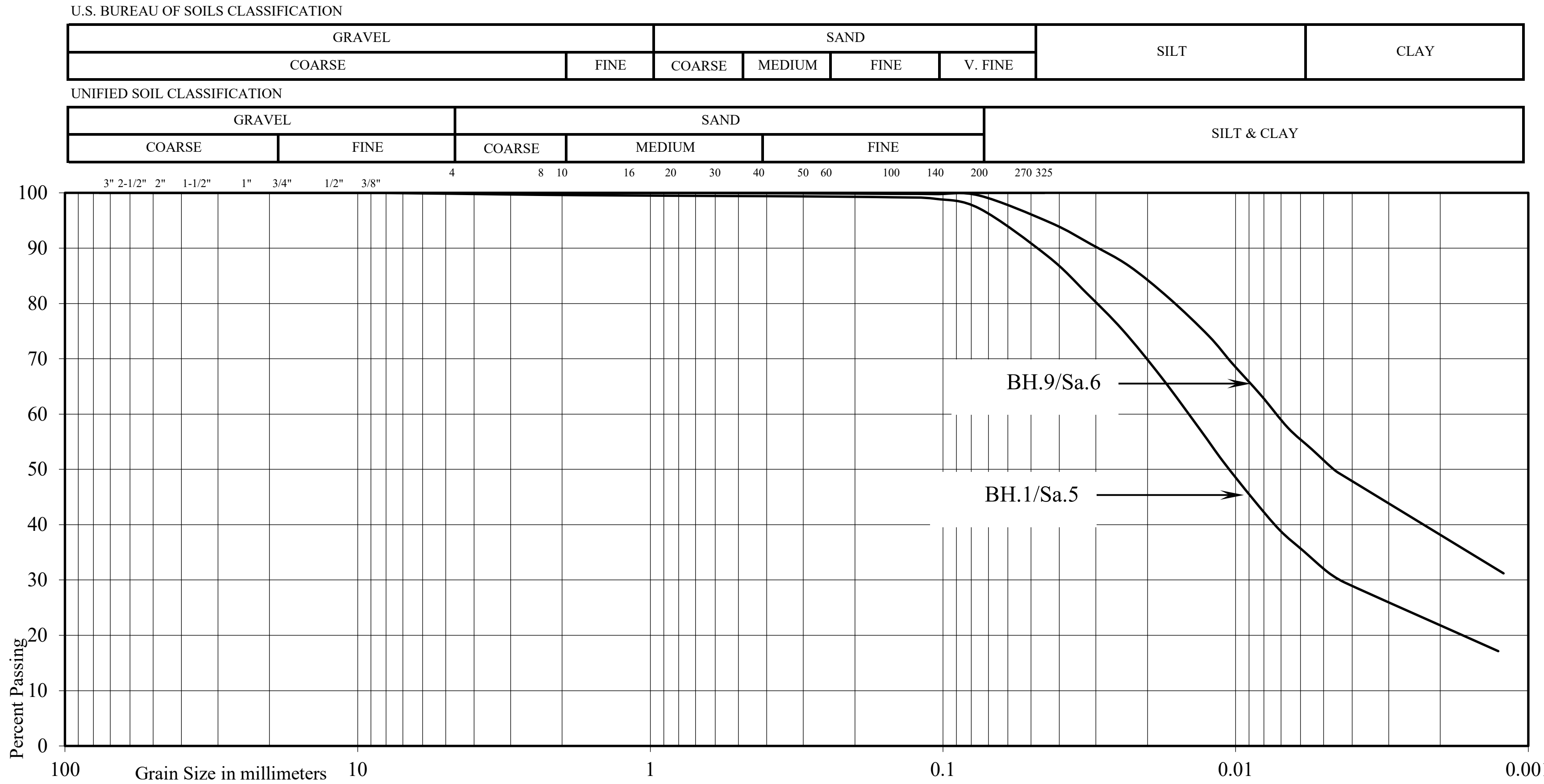
GRAIN SIZE DISTRIBUTION

Reference No: 1909-S140



Project: Proposed Residential Development

Location: 2055 Brock Road, City of Pickering

Borehole No:	1	9
Sample No:	5	6
Depth (m):	3.3	4.8
Elevation (m):	87.7	82.5

BH./Sa.	1/5	9/6
Liquid Limit (%) =	29	33
Plastic Limit (%) =	16	17
Plasticity Index (%) =	13	16
Moisture Content (%) =	24	35
Estimated Permeability (cm./sec.) =	10^{-7}	10^{-7}

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

Figure: 11

GRAIN SIZE DISTRIBUTION

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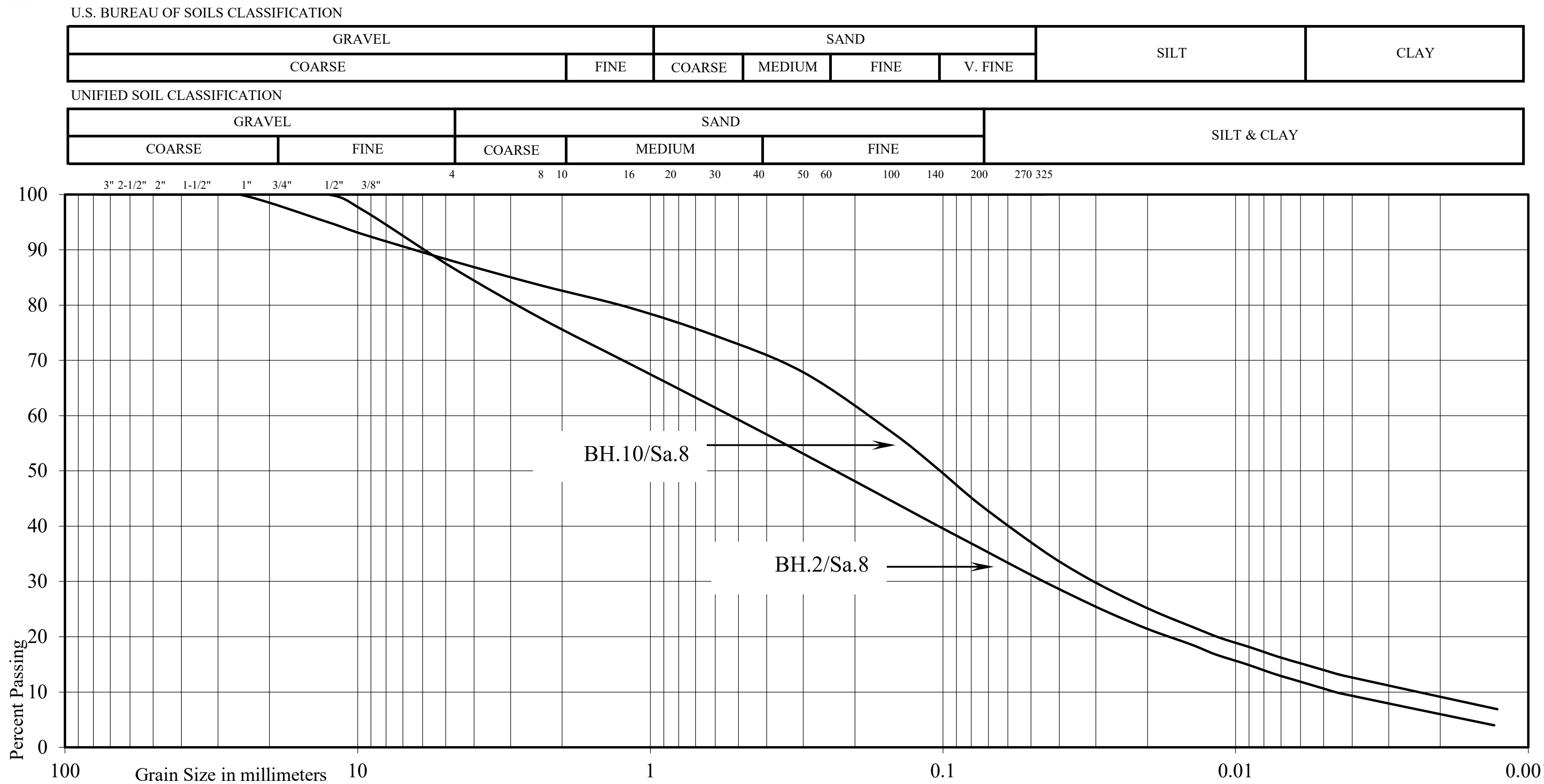
Reference No: 1909-S140



Project:	Proposed Residential Development			BH./Sa.	2/8	10/8
Location:	2055 Brock Road, City of Pickering			Liquid Limit (%) =	-	-
				Plastic Limit (%) =	-	-
Borehole No:	2	10		Plasticity Index (%) =	-	-
Sample No:	8	8		Moisture Content (%) =	6	8
Depth (m):	7.8	7.8		Estimated Permeability		
Elevation (m):	82.2	78.1		(cm./sec.) =	10^{-5}	10^{-5}

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

Figure: 12

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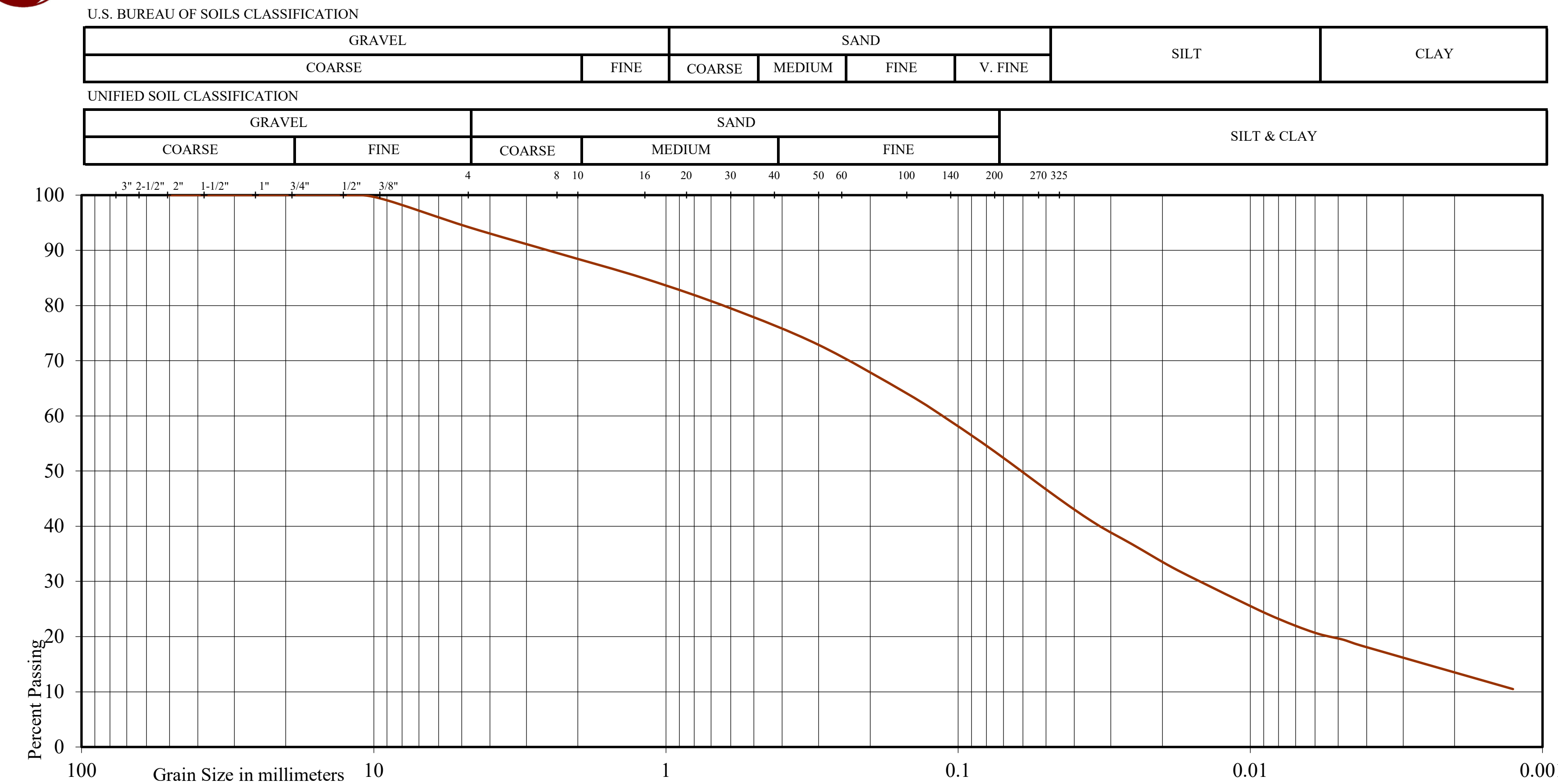
GRAIN SIZE DISTRIBUTION

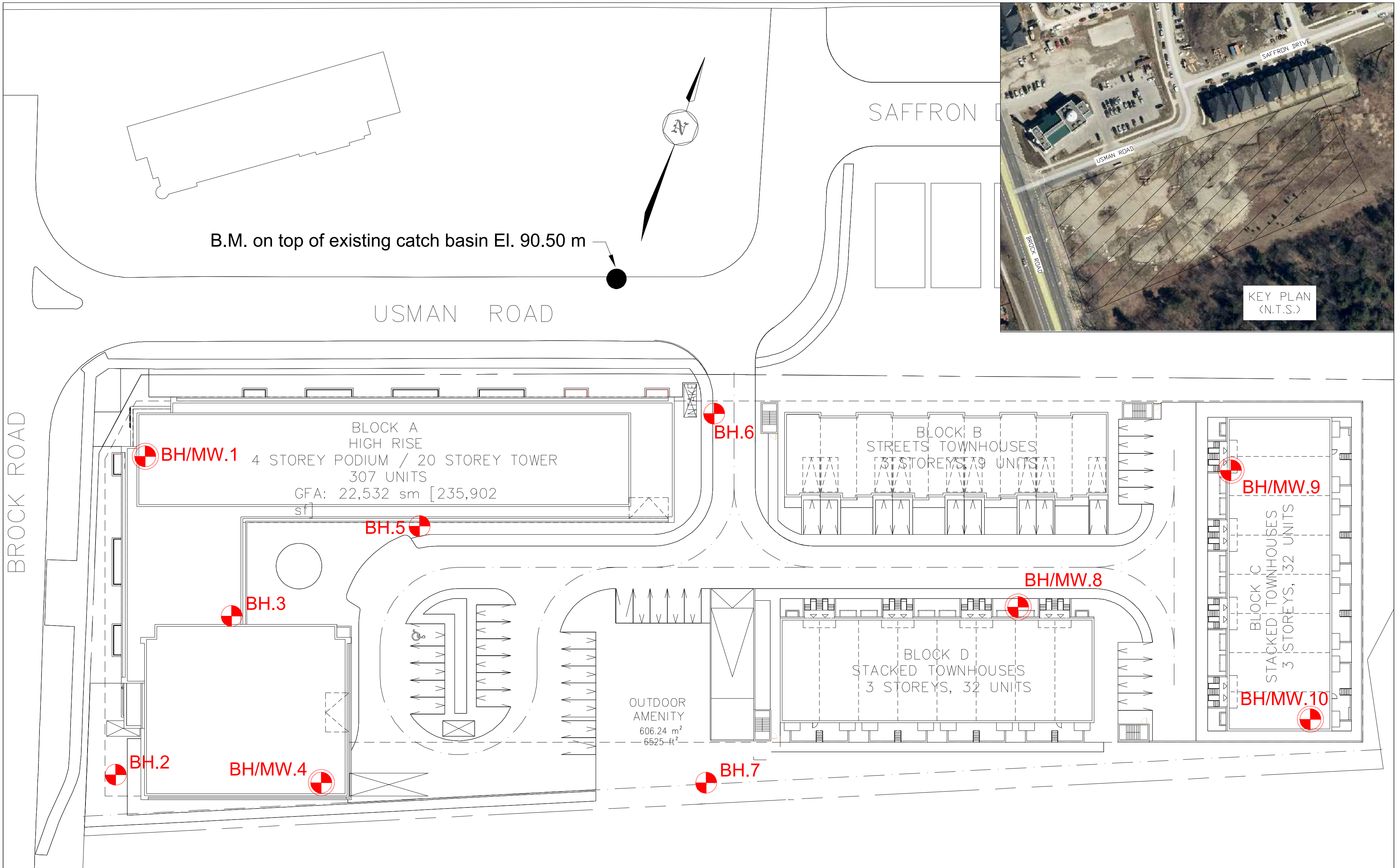
Reference No: 1909-S140



Project:	Proposed Residential Development		
Location:	2055 Brock Road, City of Pickering	Liquid Limit (%) =	-
		Plastic Limit (%) =	-
Borehole No:	6	Plasticity Index (%) =	-
Sample No:	8	Moisture Content (%) =	8
Depth (m):	7.8	Estimated Permeability	
Elevation (m):	84.0	(cm./sec.) =	10^{-6}

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

Figure: 13



LEGEND

- Borehole
- Borehole with monitoring well

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

BOREHOLE AND MONITORING WELL LOCATION PLAN

SITE: 2055 Brock Road, City of Pickering

DESIGNED BY: - **CHECKED BY:** - **DWG NO.:** 1

SCALE: 1:800 **REF. NO.:** 1910-S140 **DATE:** December 2019 **REV** -

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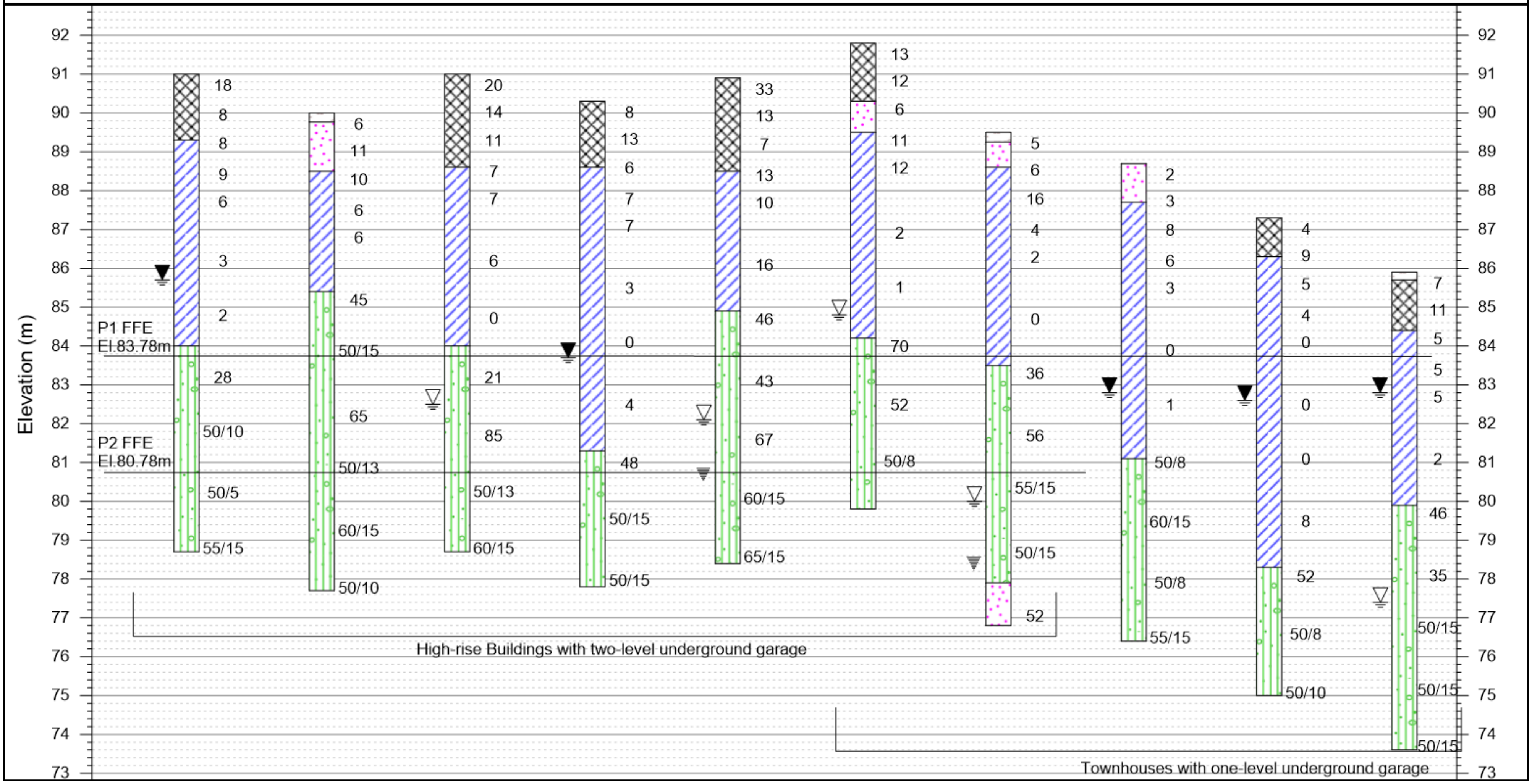
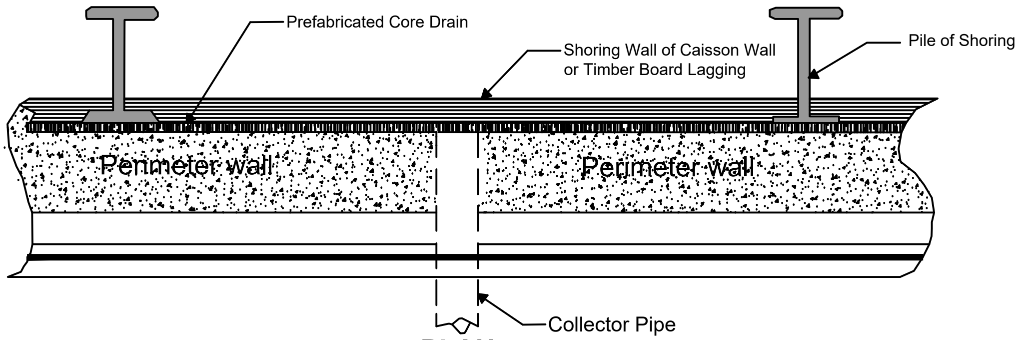
SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

JOB NO.: 1909-S140
REPORT DATE: December 2019
PROJECT DESCRIPTION: Proposed Residential Development
PROJECT LOCATION: 2055 Brock Road, City of Pickering

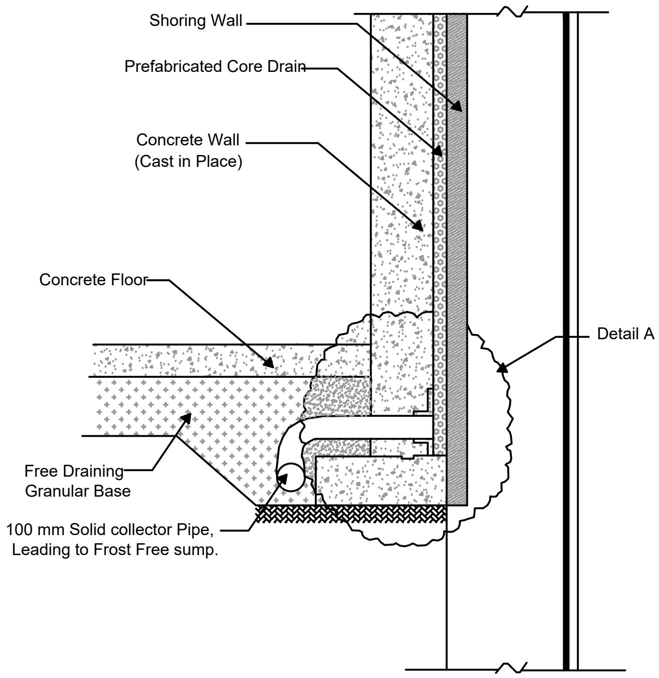
LEGEND
TOPSOIL SAND GLACIAL TILL SILTY CLAY FILL
WATER LEVEL (END OF DRILLING) CAVE-IN WATER LEVEL (STABILIZED)

BH No.:	1	2	3	4	5	6	7	8	9	10
El. (m):	91.0	90.0	91.0	90.3	90.9	91.8	89.5	88.7	87.3	85.9

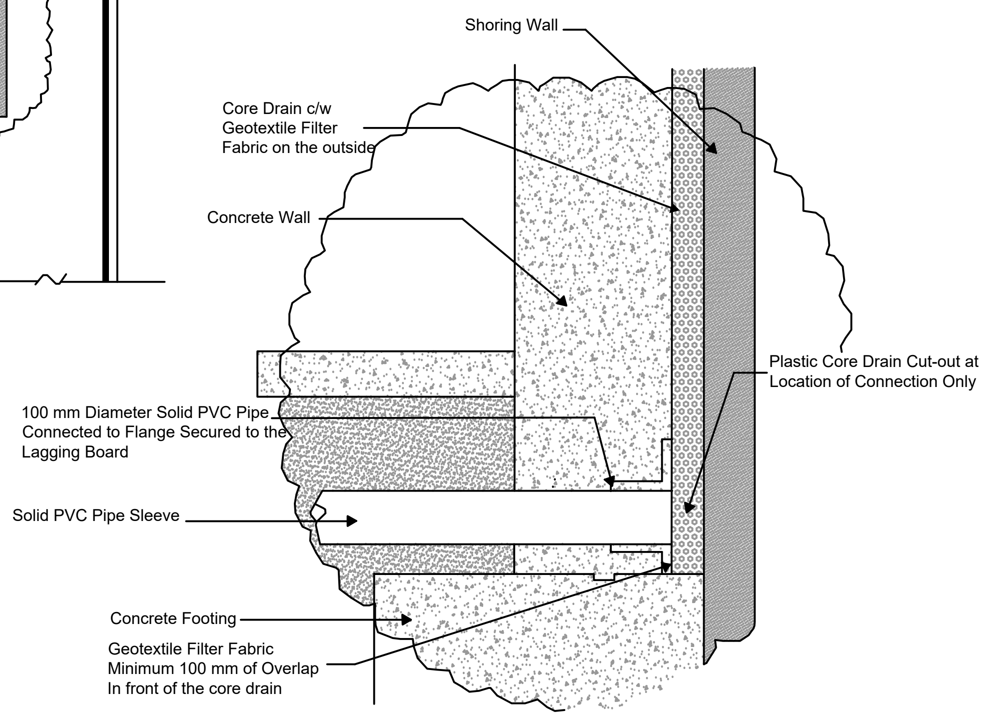




PLAN



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 □oints 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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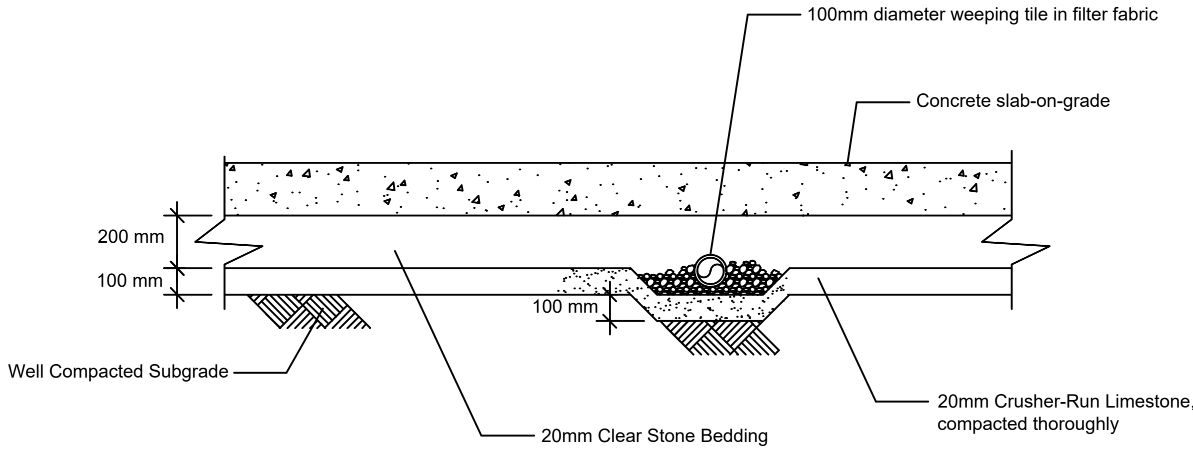
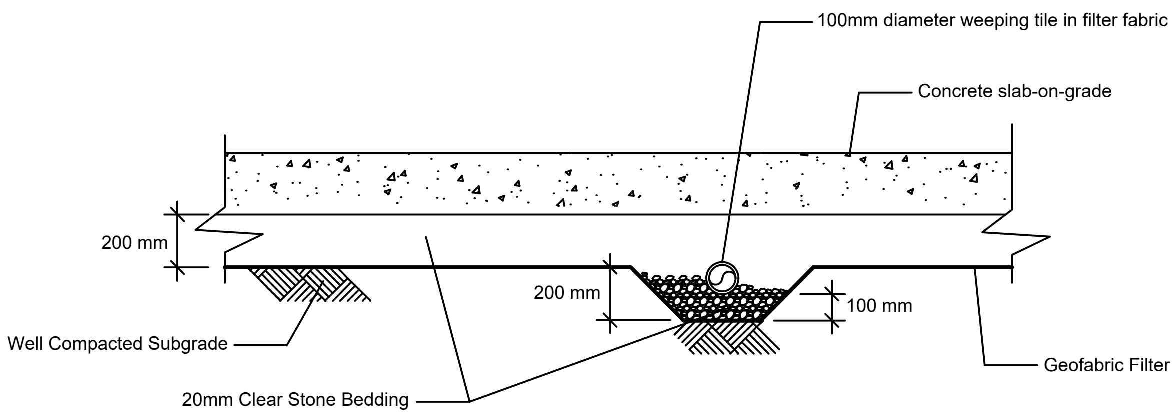
Permanent Perimeter Drainage System

SITE: 2055 Brock Road, City of Pickering

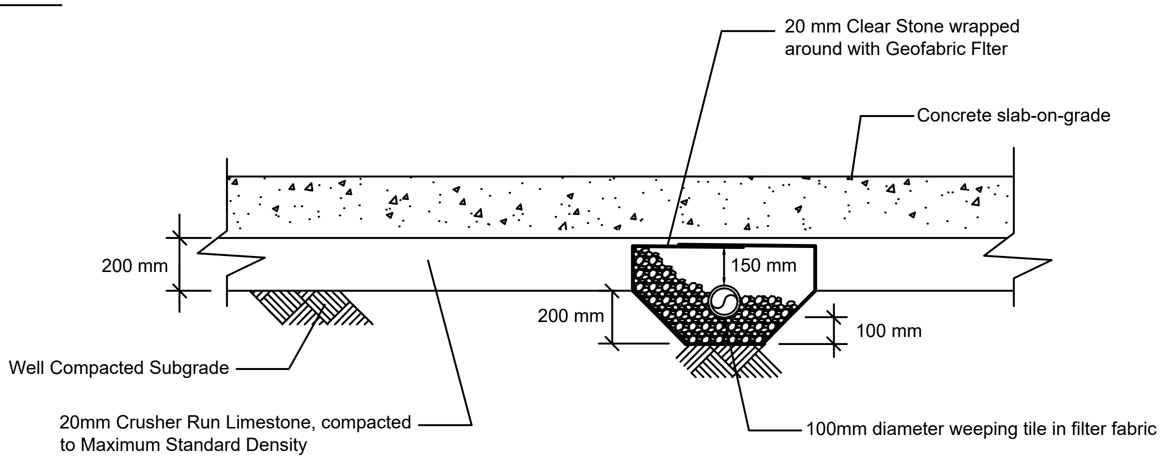
DESIGNED BY: K.L.	CHECKED BY: B.S.	DWG NO.: 3	
SCALE: N.T.S.	REF. NO.: 1909–S140	DATE: December 2019	REV -



OPTION A



OPTION B



*NOT TO SCALE

OPTION C

Note:
All weepers should drain in a positive gradient towards an outlet or a sump pit for removal by pumping

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DETAILS OF UNDERFLOOR WEEPERS

SITE: 2055 Brock Road, City of Pickering					
DESIGNED BY: K.H.		CHECKED BY: B.S.		DWG NO.: 4	
SCALE: N.T.S.	REF. NO.: 1909–S140		DATE: December 2019		REV –

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BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	GRAVENHURST	HAMILTON
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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

SHORING DESIGN

REFERENCE NO. 1909-S140

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 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 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 expressions:

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)

In Cohesive Soils: $R = 9\, c_u\, D\, (L - 1.5\, D)$

where	R = Ultimate Load to be restrained	(kN)
	D = Diameter of concrete filled hole	(m)
	L = Embedment depth of the pile	(m)
	c_u = Undrained shear strength of subsoil	(kPa)

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 clay deposit can be estimated using an adhesion value of 20 kPa. Tie back anchors extending into the till deposit can be designed using an adhesion value of 70 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 clay deposit below the bottom of excavation should be designed for the allowable bearing pressure of 50 kPa. The allowable bearing pressure of 400 kPa is recommended for rakers extending into the glacial till deposit below El. 80.0 m.

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.

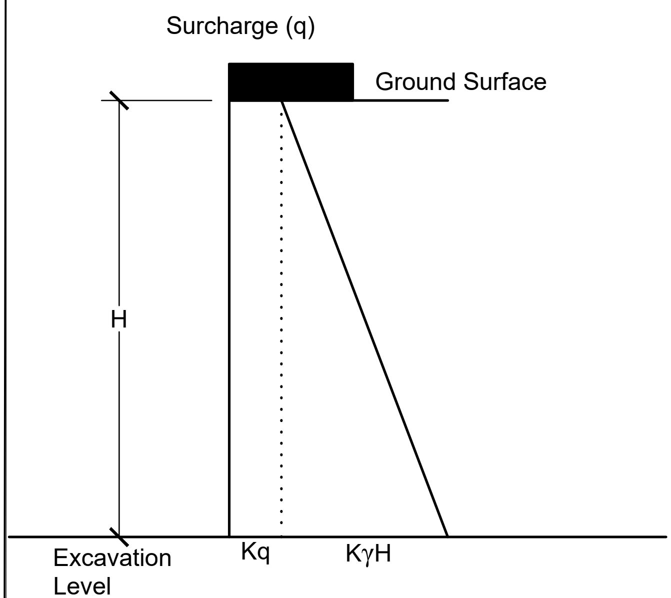
To prevent undermining of the raker footing, no excavation should be made within two times the width of raker footing on the opposite side of the raker.

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.

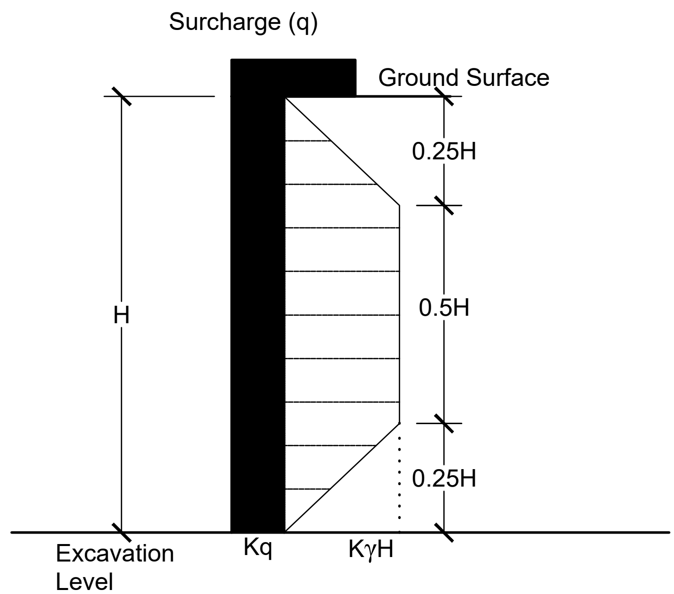
MONITORING OF PERFORMANCE

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: 2055 Brock Road, City of Pickering			
DESIGNED BY: K.L.	**CHECKED BY:** B.S.	**DWG NO.:** A1	
SCALE: N.T.S.	**REF. NO.:** 1909-S140	**DATE:** December 2019	**REV** -