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A REPORT TO
BARA GROUP (RIVER OAK) INC.

A GEOTECHNICAL INVESTIGATION FOR PROPOSED MIXED-USE BUILDINGS

2163 AND 2169 SIXTH LINE
TOWN OF OAKVILLE

REFERENCE NO. 1909-S038

NOVEMBER 2019

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1.0 INTRODUCTION

In accordance with written authorization from Mr. Khosrow Barati, President of Bara Group (River Oak) Inc. dated September 6, 2019, a geotechnical investigation was conducted at 2163 and 2169 Sixth Line in the Town of Oakville.

The purpose of the investigation was to reveal the subsurface conditions for redevelopment of the property, which will include two 6-storey mixed-use buildings with one or two levels of underground parking. The geotechnical findings and resulting recommendations are presented in this Report.

2.0 SITE AND PROJECT DESCRIPTION

The Town of Oakville is situated on Iroquois Lake plain where a drift overburden overlies shale bedrock at a shallow depth. The drift has been partly eroded and, in places, filled with lacustrine clay, silt, and sand.

The subject property is almost triangular in shape and encompasses an area of 1.8 acres. It is located at the northwest corner of Sixth Line and River Oaks Boulevard East in the Town of Oakville. At the time of investigation, the property contains single storey commercial buildings with a paved parking lot at street level. The existing site gradient is relatively flat, with drops towards the east of the property.

It is understood that the existing structures will be demolished for the construction of two 6-storey mixed-use buildings, with one or two levels of underground parking. The buildings will be comprised of retail units at ground level and residential units above.

3.0 FIELD WORK

The field work, consisting of eight (8) sampled boreholes, was performed between October 3 to 11, 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 truck-mounted, continuous-flight power-auger machine equipped for soil sampling. Standard Penetration Tests, using the procedures described on the enclosed “List of Abbreviations and Terms”, were performed at the sampling depths. The test results are recorded as the Standard Penetration Resistance (or ‘N’ values) of the subsoil. The relative density of the granular strata and the consistency of the cohesive strata are inferred from the ‘N’ values. Split-spoon samples were recovered for soil classification and laboratory testing. Conventional drilling and sampling was terminated in weathered shale at a depth of 6.1 m from the prevailing ground surface.

NQ (47.6 mm diameter) size rock coring was carried out in four selected boreholes, below the conventional drilling depth of 6.1 m and up to a depth of 8.1 to 9.4 m from grade, to assess the continuity and quality of bedrock. The rock quality has been assessed by applying the ‘Rock Quality Designation’ and unconfined compressive strength.

Upon completion of borehole drilling, soil sampling and rock coring, groundwater monitoring wells were installed in five boreholes to facilitate a hydrogeological and environmental assessment, which will be presented in other reports under separate covers.

The field work was supervised and the findings were recorded by a Geotechnical Technician. The ground elevation at each borehole location was determined using a hand-held Global Navigation Satellite System surveying equipment (Trimble Geoexplorer 6000 series).

4.0 SUBSURFACE CONDITIONS

The investigation has disclosed that beneath the pavement structure or a layer of topsoil at some locations, the site is underlain by a stratum of silty clay till overlying the shale bedrock at 2.3 to 3.3 m from the prevailing ground level. Detailed descriptions of the encountered subsurface conditions are presented on the Borehole Logs, comprising Figures 1 to 8, inclusive. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing No. 2. The engineering properties of the disclosed soils and rock are discussed herein.

4.1 **Pavement Structure** (Boreholes 1, 2, 4, 5, 7 and 8)

The existing pavement of the parking lot, as determined from the boreholes, consists of 80 to 130 mm thick asphalt, overlying 230 to 520 mm thick granular fill. Grain size analyses were performed on 3 representative samples of the granular fill; the results are plotted on Figure 9. Two of these samples were found to contain excessive fine particles (75 μm or finer), exceeding the OPS Specifications for Granular ‘A’ or Granular ‘B’. The third sample, however, meets the OPS Specifications for Granular ‘A’ and Granular ‘B’.

4.2 **Topsoil** (Boreholes 3 and 6)

A layer of topsoil, 8 cm and 13 cm in thickness, was contacted at the boreholes located in the landscaped area beside the existing buildings.

4.3 **Silty Clay Till** (All Boreholes)

The native stratum of silty clay till was contacted beneath the pavement or topsoil. Its structure is heterogeneous, consisting of a random mixture of soils particles ranging from clay to gravel, with the clay fraction exerting the dominant influence on the soil properties.

The obtained 'N' values range from 7 to more than 100, with a median of 26 blows per 30 cm of penetration, indicating the till is firm to hard, generally very stiff in consistency. Hard resistance to augering was encountered in places, showing the till is occasionally embedded with cobbles and boulders.

The natural water content of the soil samples was determined; the results range from 9% to 19%, with a median of 12%, indicating the till is in moist to very moist conditions.

Based on the above findings, the engineering properties of the clay till are given below:

- Moderately high frost susceptibility and low water erodibility.
- Low permeability, with an estimated coefficient of permeability of 10^{-7} cm/sec, a percolation rate of 80 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 clay till will be relatively stable in steep excavation; however, under prolonged exposure, the sand seams may become saturated, leading to localized sloughing.
- A poor pavement-supportive material, with an estimated California Bearing Ratio (CBR) value of 5%.
- Moderately high corrosivity to buried metal, with an estimated electrical resistivity of 3000 ohm·cm.

4.4 **Shale Bedrock** (All Boreholes)

Shale bedrock was contacted below the clay till at a depth of ranging from 2.3 to 3.3 m from the prevailing ground surface. It is reddish brown in colour, indicating Queenstone Formation. The shale is thinly bedded, predominantly of mudstone with occasional hard, limy shale. It is susceptible to disintegration and swelling upon exposure to air and water, with subsequent reversion to a clay soil.

The shale can be penetrated by power-augering with some difficulty in grinding through the hard layers. It is mostly fissured as a result of weathering and/or overstressing by glaciation. Infiltrated precipitation and groundwater from the overburden soils will often permeate the fissures in the rock and, in places, will be under subterranean artesian pressure. However, because the shale is a clay rock, it is considered to be a material of low permeability and a poor aquifer, and the groundwater yield from the rock will be limited. The water content values of the shale fragments or rock dust samples were 4% to 6%.

NQ size rock cores were collected from Boreholes 1, 2, 7 and 8, below a depth of 6.1 m. The quality and the soundness of bedrock is determined by interpreting the rate of Recovery (RC) and the Rock Quality Designation (RQD) of the rock cores, as presented on the borehole logs.

Two relatively sound rock specimens were selected for Unconfined Compression Test (CSA A23.2-14C). The test results are summarized in Table 1.

Table 1 - Uniaxial Compressive Strength of Rock Core Samples

Sample Location	Borehole 2 Depth 6.1 m	Borehole 8 Depth 8.1 m
Elevation (m)	142.5	138.8
Density (kg/m^3)	2613	2590
Compressive Strength (MPa)	20.0	17.2

The shale bedrock can be classified as “Weak Rock”. At the coring depths below 6.1 m, the bedrock is in a very poor to good quality, having the RQD ranging from 16% to 88%. Weathered shale is considered to extend to a depth of 5.5 to 6.5 m from grade.

The shale is susceptible to disintegration and swelling upon exposure to air and water, with subsequent reversion to a clay soil, but the laminated limy and sandy layers would remain as rock slabs. Weathered shale can be excavated with considerable effort by a heavy-duty backhoe equipped with a rock-ripper; however, excavation will become progressively more difficult with depth into the sound shale. Efficient removal of the sound shale may require the aid of pneumatic hammering.

The excavated spoil will contain a large amount of hard limy rock slabs, rendering it virtually impossible to obtain uniform compaction. Therefore, unless the spoil is sorted, it is considered unsuitable for reuse as backfill material or engineering applications.

When excavating into the sound shale, slight lateral displacement of the excavation wall is often experienced. This is due to the release of residual stress stored in the bedrock mantle and the swelling characteristic of the rock.

5.0 GROUNDWATER CONDITIONS

No groundwater or cave-in occurred in the open boreholes throughout the investigation period. The groundwater levels recorded in the monitoring wells on October 10, 21 and 31, 2019, are summarized in Table 2.

Table 2 - Stabilized Groundwater Level in Monitoring Wells

Borehole/ Well No.	Well Depth (m)	Ground Elevation (m)	October 10, 2019		October 21, 2019		October 31, 2019	
			Depth (m)	**El. (m)**	**Depth (m)**	**El. (m)**	**Depth (m)**	**El. (m)**
BH/MW 1	9.1	148.6	5.3	143.3	5.3	143.3	5.2	143.4
BH/MW 2	9.1	148.6	5.8	142.8	5.6	143.0	5.1	143.5
BH/MW 4	6.1	147.5	5.2	142.3	Dry	-	Dry	-
BH/MW 7	9.0	147.0	5.7	141.3	5.5	141.5	5.3	141.7
BH/MW 8	9.1	146.9	6.0	140.9	5.9	141.0	5.7	141.2

The recorded groundwater level in the monitoring wells ranges from 5.1 to 6.0 m below grade, or between El. 140.9 m and El. 143.5 m. The water level represents perched groundwater in the fractured bedrock. It is anticipated to fluctuate under seasonal conditions.

In excavation, the groundwater yield from the overburden is anticipated to be slow in rate and limited in quantity. In fractured rock, the fissures may contain groundwater under subterranean artesian pressure and the yield can be initially moderate and persistent. Upon release through excavation, the water is expected to drain readily and stop after a short period of time.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation has disclosed that beneath the pavement structure or a layer of topsoil, the site is underlain by a stratum of firm to hard silty clay till, overlying the shale bedrock at a depth of 2.3 to 3.3 m from the prevailing ground level.

The shale bedrock is reddish brown in colour, indicating Queenstone Formation. It can be classified as "Weak Rock", in a very poor to good quality. Weathered shale is considered to extend to a depth of 5.5 to 6.5 m from grade. It is susceptible to disintegration and swelling upon exposure to air and water, with subsequent reversion to a clay soil.

No groundwater or cave-in occurred in the open boreholes throughout the investigation period. However, groundwater was recorded in some monitoring wells between El. 140.9 m and El. 143.5 m, representing the perched groundwater in the fractured bedrock.

It is understood that the existing structures will be demolished for the construction of two 6-storey mixed-use buildings, with one or two levels of underground parking. The buildings will be comprised of retail units at ground level and residential units above. The geotechnical findings which warrant special consideration are presented below:

1. After demolition and removal of the existing structures, the cavities must be backfilled with selected on-site material, compacted properly in layers for the preparation of a construction platform for the access of shoring and construction equipment.
2. Shoring will be required for excavation of the underground structure if a safe backing slope is not possible.
3. Excavations into the shale will require extra effort using an excavator equipped with a rock ripper. Pneumatic hammering will be required to break up the rock mass for efficient rock excavation.
4. Slight lateral displacement of the excavation walls is often experienced in sound rock, due to the release of residual stress in the bedrock mantle and the swelling characteristics of the rock. It is necessary to place concrete for the footings immediately after it is exposed and inspected. If the excavation will be left open for some time, a mud slab of lean mix concrete must be placed on the bearing surface to prevent disintegration and swelling of the shale.
5. The foundation subgrade must be inspected by a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to ensure that its condition is compatible with the design of the foundation.
6. Perimeter subdrains and dampproofing of the foundation walls will be required for the underground structure. The subdrains should be shielded by a fabric filter to prevent blockage by silting.
7. A pre-construction survey is recommended for the adjacent properties and structures prior to any excavation and construction activities at the site.

The recommendations appropriate for the project described in Section 2.0 are presented herein. One must be aware that the subsurface conditions may vary between boreholes.

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

6.1 **Foundations**

The proposed buildings will be provided with one level or 2 levels of underground parking. The excavation for the foundation is anticipated to extend into the shale bedrock below 3 m from the ground surface. The recommended Maximum Allowable Soil Bearing Pressure (SLS) and the Factored Ultimate Soil Bearing Pressure (ULS) for the design of conventional footings founded on shale bedrock are given below:

- One-Level Underground Parking – assumed foundation founding El. 144 to 145 m:
  - Maximum Allowable Soil Bearing Pressure (SLS) = 1200 kPa
  - Factored Ultimate Soil Bearing Pressure (ULS) = 2000 kPa

  The total and differential settlements of foundation, designing for the Maximum Allowable Bearing Pressure (SLS), are estimated within 25 mm and 20 mm, respectively.

- Two-Level Underground Parking – assumed foundation founding El. 141 to 142 m:
  - Maximum Allowable Soil Bearing Pressure (SLS) = 2000 kPa
  - Factored Ultimate Soil Bearing Pressure (ULS) = 3000 kPa

  The anticipated total and differential settlement of footings will be insignificant.

The foundation subgrade should be clear of disturbed material or loose debris. For footings bearing on sound shale below 6 m from grade, rock layers having close spacing of discontinuity (less than 0.1 m) should be subexcavated. The bearing surface 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.

The shale bedrock will slake if left exposed for any length of time. It is, therefore, important that the footings are poured with concrete immediately on excavation and inspection. If the footing area will be left open for some time, the footings should be skim coated with lean mix concrete to minimize deterioration of rock at the bearing surface.

Shale is considered as frost susceptible. Footings exposed to weathering, or in unheated areas, should have at least 1.2 m of earth cover for protection against frost action. For an unheated underground parking garage with limited open access, a minimum earth cover of 0.9 m for interior footings and 0.6 m for perimeter footings is necessary for frost protection.

Footings adjacent to the fresh air ducts, the entrance of the garage and other areas which may be exposed to the extreme temperature from the exterior should be provided with a minimum frost cover of 1.2 m or properly insulated.

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

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

6.2 Underground Structure

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

In areas where the perimeter walls extend into the sound shale, a compressible material, such as sprayed foam, 100 mm in thickness, should be placed between the concrete wall and the bedrock, or the excavation should provide at least 400 mm space between the bedrock face and the foundation walls to be filled with loose sand afterwards. This is to allow lateral expansion or movement of the rock face without causing damage to the foundation walls.

In conventional design of underground structures, the perimeter walls should be dampproofed and provided with a perimeter subdrain at the wall base as shown in Drawing No. 3. Backfill of open excavation should be free-draining granular material unless prefabricated drainage board is installed over the entire wall below grade. At the shoring location, prefabricated drainage board, such as Miradrain 6000 or equivalent, must be provided on the perimeter walls, between the shoring wall or rock face and the cast-in-place foundation wall, as shown in Drawing No. 4. The perimeter subdrains should be installed on a positive gradient, connecting into the frost-free sump-well and discharge into the storm sewers. All the subdrains should be encased in a fabric filter to protect them against blockage by silting.

If the municipality does not allow the connection of subsurface water draining into the sewer system, a cistern will have to be provided to retain the subsurface water for other uses, such as irrigation or surface cleaning at the site. If a cistern is not feasible, the underground structure will have to be waterproofed and designed to resist the hydrostatic pressure on the walls and floor.

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

The ground surface around the buildings must be graded to direct water away from the structure.

6.3 **Slab-On-Grade Construction**

The subgrade for conventional slab-on-grade construction should consist of undisturbed native soil, bedrock, or properly compacted inorganic earth fill. In preparation of the subgrade, any weathered soil and deleterious material must be removed and replaced with properly compacted organic free earth fill or the bedding material.

The slab-on-grade should be constructed on a granular base of 20-mm Crusher-Run Limestone, not less than 20 cm thick and compacted to its maximum Standard Proctor dry density. A Modulus of Subgrade Reaction of 35 MPa/m can be used for the design of the floor slab.

At the garage entrances, the subgrade should be properly insulated, or the subgrade material should be replaced with 1.0 m of non-frost-susceptible granular material and should be provided with subdrains. This will minimize frost action in this area where vertical ground movement cannot be tolerated. The floor at the entrance and in areas of close proximity to air shafts should be insulated, and the insulation should extend 1.5 m internally. This measure is to prevent frost action induced by cold drafts.

The exterior grade should slope away from the building. This is to prevent runoff from ponding in the areas adjacent to the underground structure.

6.4 **Underground Services**

The subgrade for the underground services should consist of undisturbed native soils, bedrock or compacted organic-free earth fill. Where the subgrade consists of badly weathered soils, they must be subexcavated and replaced with properly compacted bedding material.

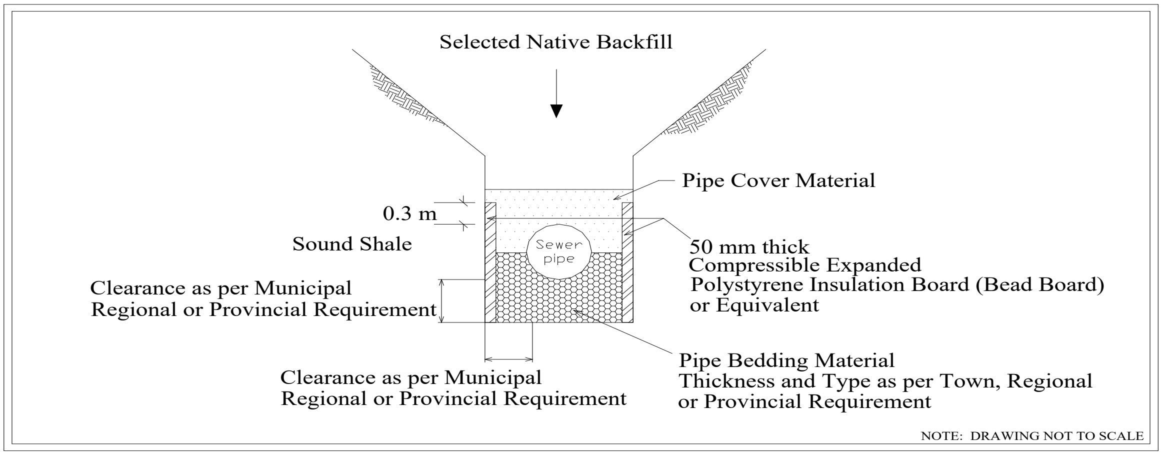
A Class ‘B’ bedding, consisting of compacted 20-mm Crusher-Run Limestone, is recommended for the construction of the underground services. The sewer joints should be leak-proof, or wrapped with an appropriate waterproof membrane to prevent subgrade migration.

In order to prevent pipe flotation 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 with a fabric filter to prevent blockage by silting.

If excavation into the sound shale is required, the sides should be sloped rather than vertical, due to the residual stress relief and the swelling characteristics of the shale. The side slopes should be no steeper than 2 vertical:1 horizontal. Alternatively, vertical trench walls can be lined with a cushioning foam layer and backfilled with sand up to 0.3 m above the crown of the pipe and flooded. The recommended scheme is illustrated in Diagram 1.

Diagram 1 - Sewer Installation in Sound Shale



The on site native soils have a moderately high corrosivity to buried metal; therefore, the underground services should be protected against soil corrosion. For estimation of anode weight requirements, the estimated electrical resistivity of 3000 ohm·cm can be used. This, however, should be confirmed by testing the soil along the trench at the time of construction.

6.5 Backfilling in Trenches and Excavated Areas

The structural backfill should be compacted on the wet side of the optimum to at least 95% of its maximum Standard Proctor density and shall be increased to 98% below the floor slab. In the zone within 1.0 m below the road subgrade, the backfill should be compacted with the water content 2% to 3% drier than the optimum to at least 98% of its maximum Standard Proctor dry density.

In normal project construction practice, the problem areas of settlement largely occur adjacent to foundation walls, columns, manholes, catch basins and services crossings. In areas which are inaccessible to a heavy compactor, the interface of the native soils and sand backfill will have to be flooded for a few days.

The narrow trenches for service crossing should be cut at 1 vertical:2 horizontal so that the backfill in the trenches can be effectively compacted. Otherwise, soil arching in the trenches will prevent achievement of the proper compaction. 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.

6.6 Pavement Design

Where the pavement is to be built on structural slabs such as the underground parking structure, sufficient granular base and adequate drainage must be provided to prevent frost damage to the pavement. An impervious 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 underground structure is presented in Table 3.

Table 3 - Pavement Design (Roof of Underground Garage)

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	60	HL-8
Granular Base	200	20-mm Crusher-Run Limestone
Granular Sub-base	100	Free-draining Sand Fill

For on-grade portion of the parking lot and driveways, the recommended pavement structure 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		HL-8
Light-Duty Parking	50	
Fire Route	60	
Granular Base	150	20-mm Crusher-Run Limestone
Granular Sub-base		
Light-Duty Parking	200	50-mm Crusher-Run Limestone
Fire Route	300	

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

The weathered soils and any soft/loose subgrade must be subexcavated, sorted free of any deleterious materials, aerated and properly compacted. If the deleterious materials cannot be sorted, the soils should be replaced by properly compacted, organic-free earth fill or granular materials. Earth fill used to raise the grade for pavement construction should consist of organic-free soil uniformly compacted to 95% or + of its 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.

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

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

6.7 Sidewalks, Interlocking Stone Pavement and Landscaping

Interlocking stone pavement, sidewalks and landscaping structures in open areas should be designed to tolerate the frost-induced ground movement. In areas where ground movement is not tolerable, such as in front of building entrances, the sidewalk 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 sidewalk, 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, the area can be properly insulated with 50-mm Styrofoam, or equivalent.

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

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**
Granular Fill/Earth Fill	21.5	1.25	0.95
Silty Clay Till	22.5	1.35	1.05
Shale Bedrock	26.0	1.50	1.30

Lateral Earth Pressure Coefficients	**Active K_a**	**At Rest K_o**	**Passive K_p**
Compacted Earth Fill	0.35	0.50	3.00
Silty Clay Till/weathered Shale	0.30	0.45	3.30
Sound Shale	0.20	0.30	5.00

Coefficients of Friction	
Between Concrete and Granular Base	0.50
Between Concrete and Natural Soils or Shale Bedrock	0.35

6.9 Excavation

Excavation should be carried out in accordance with Ontario Regulation 213/91. The types of material are classified in Table 6.

Table 6 - Classification of Material for Excavation

Material	Type
Sound Bedrock	1
Silty Clay Till/weathered Shale	2
Earth Fill/weathered Soil	3

Where sloped excavation is not feasible, a braced shoring will be required. The overburden load and the surcharge from any adjacent structures should be considered in the design of shoring. Recommendations for the shoring design are provided in the Appendix.

Any excavation into the shale will require considerable effort with a heavy-duty backhoe equipped with a rock-ripper; however, excavation will become progressively more difficult with depth into the sound shale. Efficient removal of the sound shale will require the aid of pneumatic hammering.

The shale is susceptible to disintegration and swelling upon exposure to air and water, with subsequent reversion to a clay soil. Slight lateral displacement of the excavation walls is often experienced. This is due to the release of residual stress stored in the bedrock mantle and the swelling characteristic of the rock.

In excavation, seepage from continuous groundwater is not anticipated. Any groundwater yield from the percolation of surface water may be collected into sumps and removed by pumping. In fractured rock, the fissures may contain groundwater under subterranean artesian pressure and the yield can be initially moderate and persistent. Upon release through excavation, the water is expected to drain readily and stop after a short period of time.

6.10 Monitoring of Performance

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

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

7.0 LIMITATION OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of Bara Group (River Oak) Inc. and for review by its designated consultants and government agencies. Use of this report is subject to the conditions and limitations of the contractual agreement.

The material in the report reflects the judgement of Nislin Andani, B.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, or any reliance on decisions to be made based on it, is 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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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.: 1909-S038

LOG OF BOREHOLE NO.: BH/MW 1

FIGURE NO.: 1

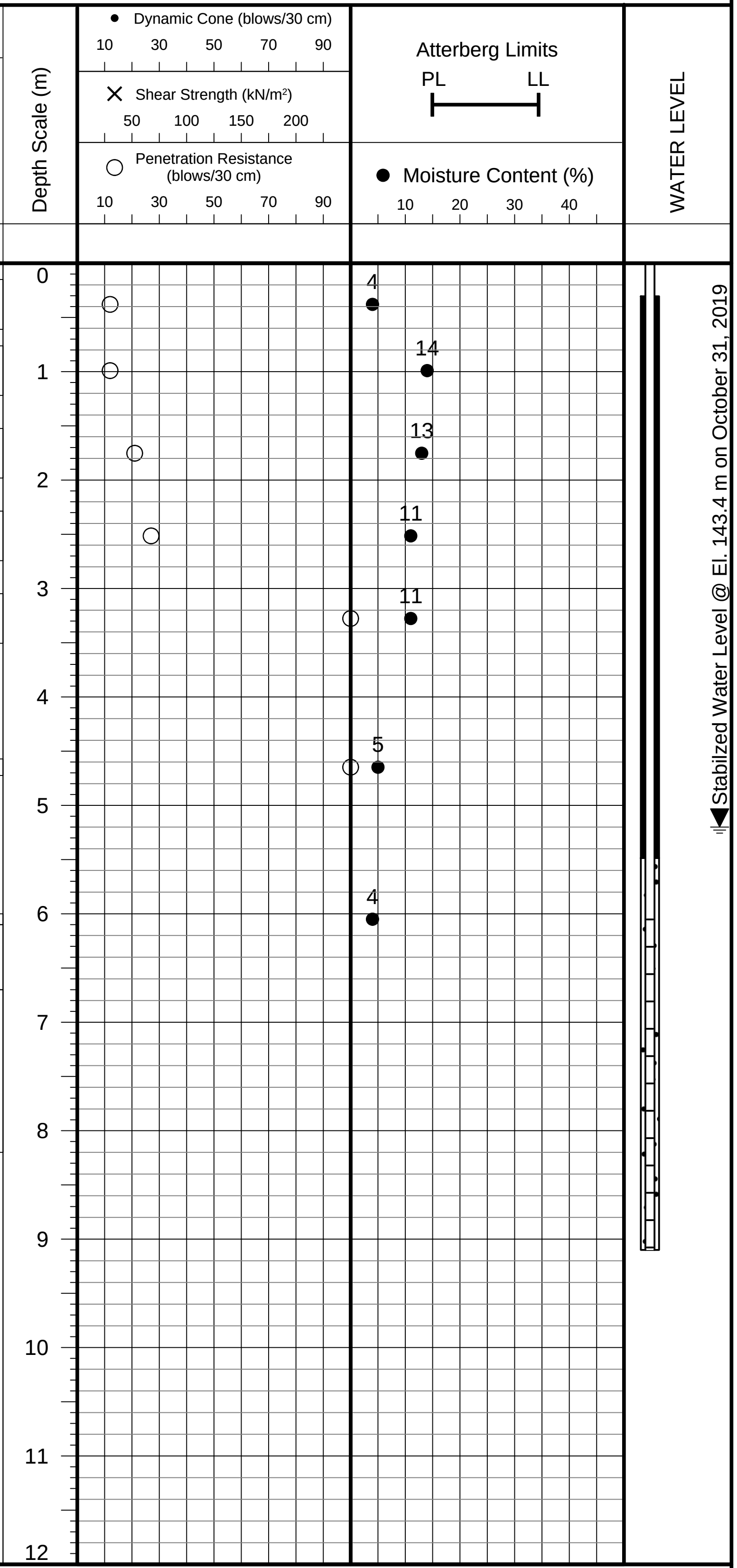
PROJECT DESCRIPTION: Proposed Mixed-Use Buildings

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2163 and 2169 Sixth Line, Town of Oakville

DRILLING DATE: October 3, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
148.6	Pavement Surface			
0.0	**10 cm ASPHALT** **50 cm GRANULAR FILL**	1	DO	12
	Reddish brown, stiff to hard weathered **SILTY CLAY TILL** a trace of gravel occ. sand seams and layers, cobbles and boulders	2	DO	12
		3	DO	21
		4	DO	27
145.4 3.2	Reddish brown, hard **SHALE** stratified weathered to 6.5 m	5	DO	50/10
		6	DO	50/15
		7	AS	-
		8	RC RQD	58% 16%
		9	RC RQD	100% 50%
140.4 8.2	**END OF BOREHOLE** Re-drilled borehole to 9.1 m Installed 50 mm Ø monitoring well to 9.1 m 3.0 m screen from 6.1 to 9.1 m Sand backfill from 5.5 to 9.1 m Bentonite seal from 0.3 to 5.5 m Provided with a Flush Mount Casing			



Stabilized Water Level @ El. 143.4 m on October 31, 2019

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JOB NO.: 1909-S038

LOG OF BOREHOLE NO.: BH/MW 2

FIGURE NO.: 2

PROJECT DESCRIPTION: Proposed Mixed-Use Buildings

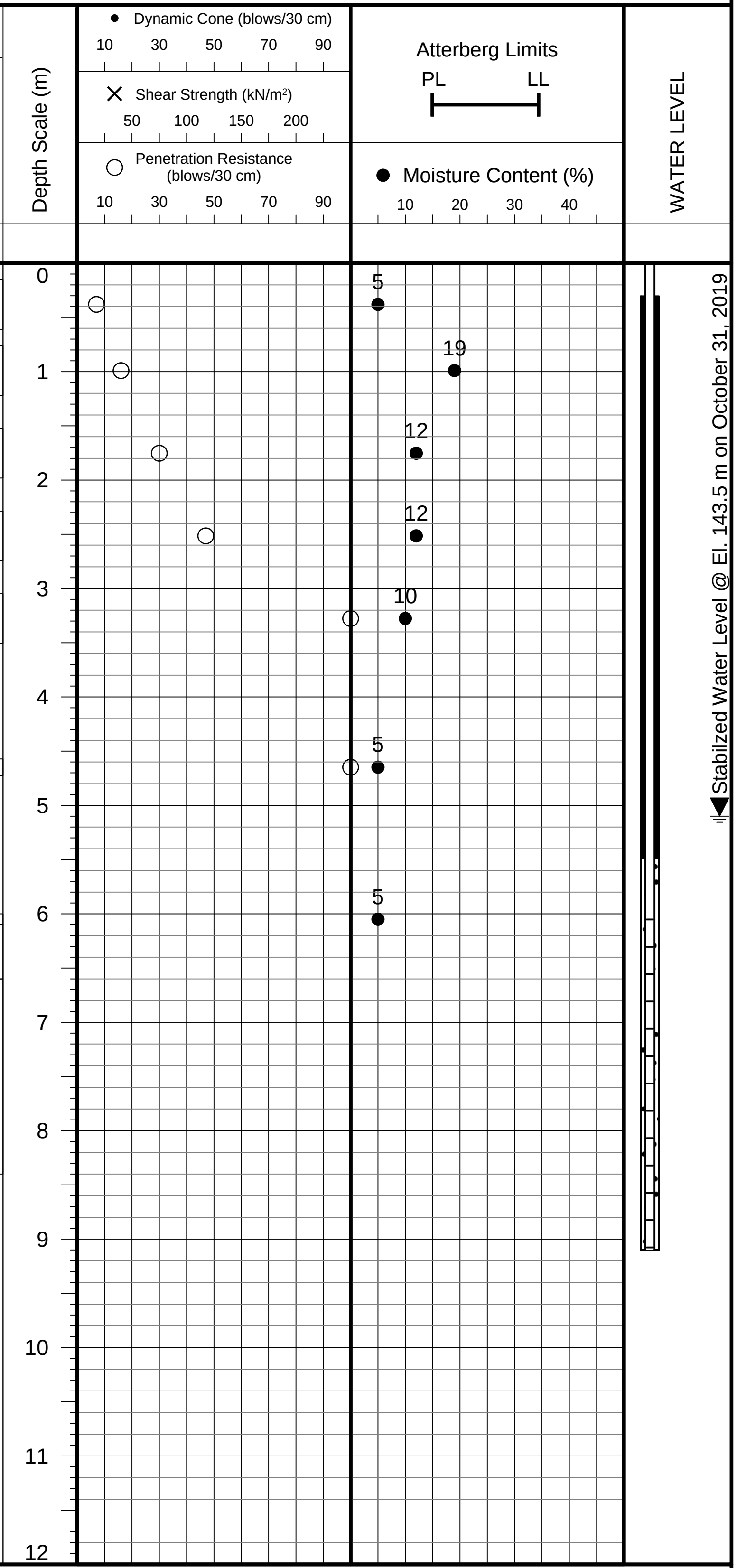
METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2163 and 2169 Sixth Line, Town of Oakville

DRILLING DATE: October 10, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
148.6	Pavement Surface			
0.0	**10 cm ASPHALT** **50 cm GRANULAR FILL**	1	DO	7
	Reddish brown, very stiff to hard **SILTY CLAY TILL** a trace of gravel occ. sand seams and layers, cobbles and boulders	2	DO	16
		3	DO	30
		4	DO	47
145.3 3.3	Reddish brown, hard **SHALE** stratified weathered to 6 m	5	DO	50/13
		6	AS	NP
		7	AS	-
		8	RC RQD	100% 70%
		9	RC RQD	98% 62%
140.2 8.4	**END OF BOREHOLE**			

Re-drilled borehole to 9.1 m
Installed 50 mm Ø monitoring well to 9.1 m
3.0 m screen from 6.1 to 9.1 m
Sand backfill from 5.5 to 9.1 m
Bentonite seal from 0.3 to 5.5 m
Provided with a Flush Mount Casing



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JOB NO.: 1909-S038

LOG OF BOREHOLE NO.: 3

FIGURE NO.: 3

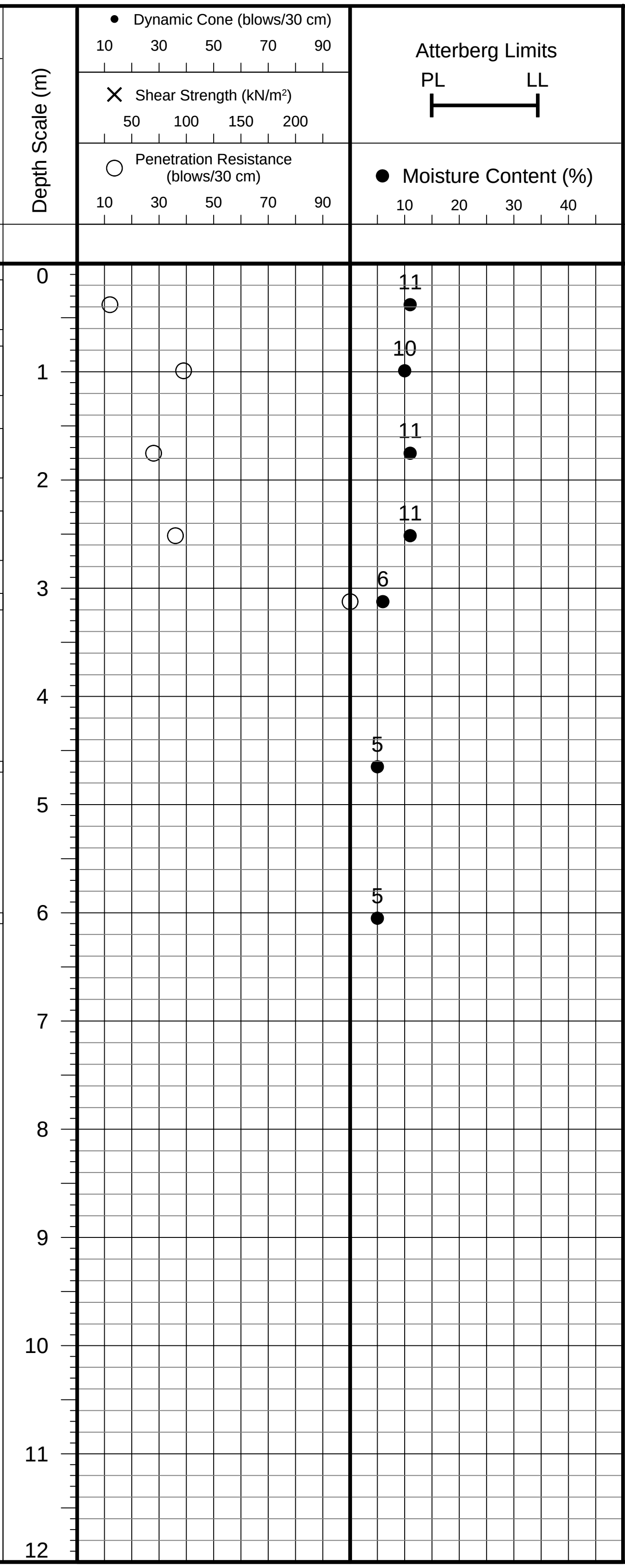
PROJECT DESCRIPTION: Proposed Mixed-Use Buildings

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2163 and 2169 Sixth Line, Town of Oakville

DRILLING DATE: October 10, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value	WATER LEVEL
147.5	Pavement Surface				
0.0	**8 cm TOPSOIL**				Dry on completion
	Reddish brown, stiff to hard weathered **SILTY CLAY TILL** a trace of gravel occ. sand seams and layers, cobbles and boulders	1	DO	12	
		2	DO	39	
		3	DO	28	
		4	DO	36	
144.5 3.0	Reddish brown, hard **SHALE** stratified, weathered occ. clay seams and limestone layers	5	DO	50/5	
		6	AS	-	
		7	AS	-	
141.4 6.1	**END OF BOREHOLE**				



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JOB NO.: 1909-S038

LOG OF BOREHOLE NO.: BH/MW 4

FIGURE NO.: 4

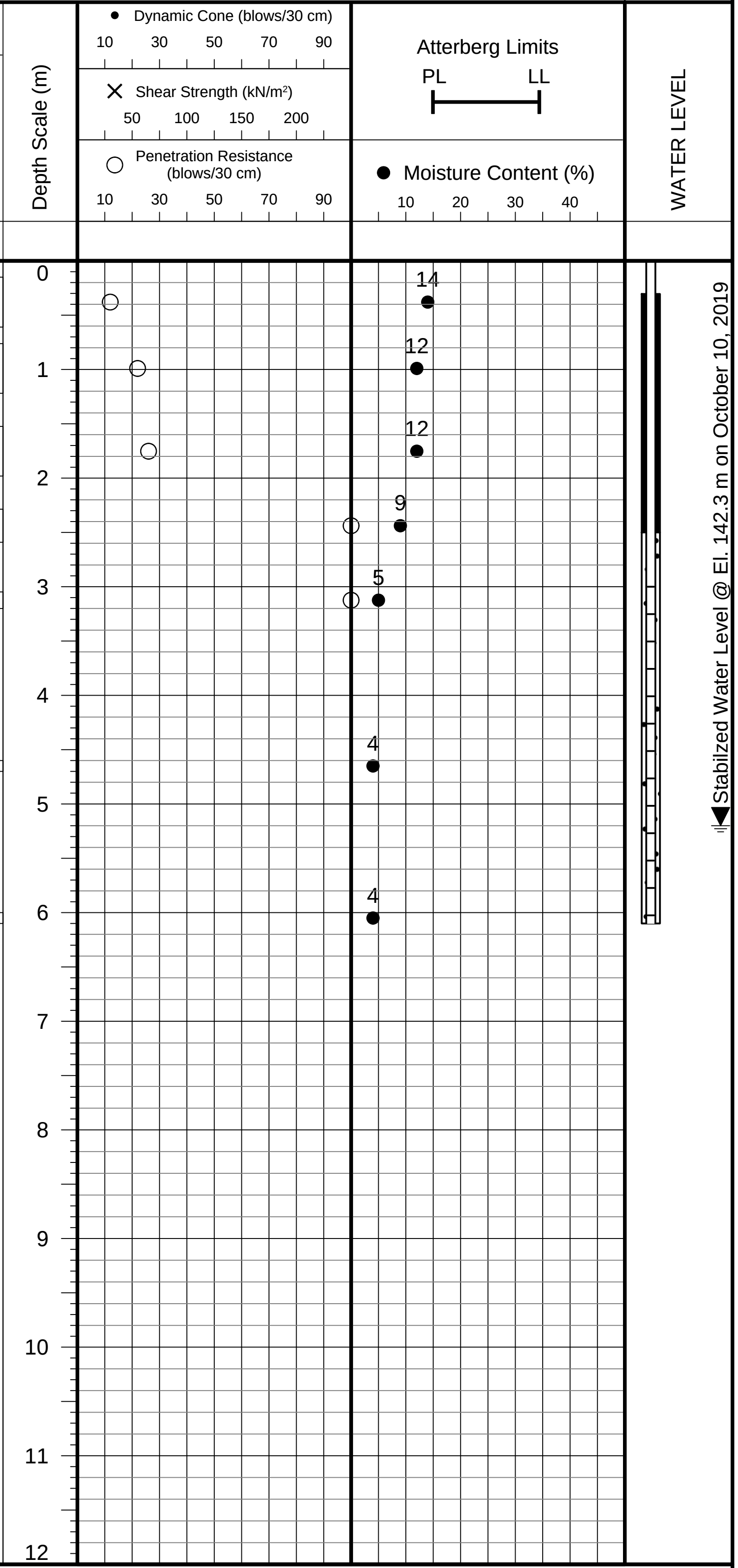
PROJECT DESCRIPTION: Proposed Mixed-Use Buildings

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2163 and 2169 Sixth Line, Town of Oakville

DRILLING DATE: October 11, 2019

		SAMPLES		
El. (m) Depth (m)	SOIL DESCRIPTION	Number	Type	N-Value
147.5	Pavement Surface			
0.0	**8 cm ASPHALT** **23 cm GRANULAR FILL**			
	weathered Reddish brown, stiff to very stiff **SILTY CLAY TILL** a trace of gravel occ. sand seams and layers, cobbles and boulders	1	DO	12
		2	DO	22
		3	DO	26
145.2 2.3	Reddish brown, hard **SHALE** stratified, weathered occ. clay seams and limestone layers	4	DO	50/10
		5	DO	50/15
		6	AS	-
		7	AS	-
141.4 6.1	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 6.1 m, 3.0 m screen from 3.1 to 6.1 m Sand backfill from 2.5 to 6.1 m Bentonite seal from 0.3 to 2.5 m Provided with a Flush Mount Casing			



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JOB NO.: 1909-S038

LOG OF BOREHOLE NO.: 5

FIGURE NO.: 5

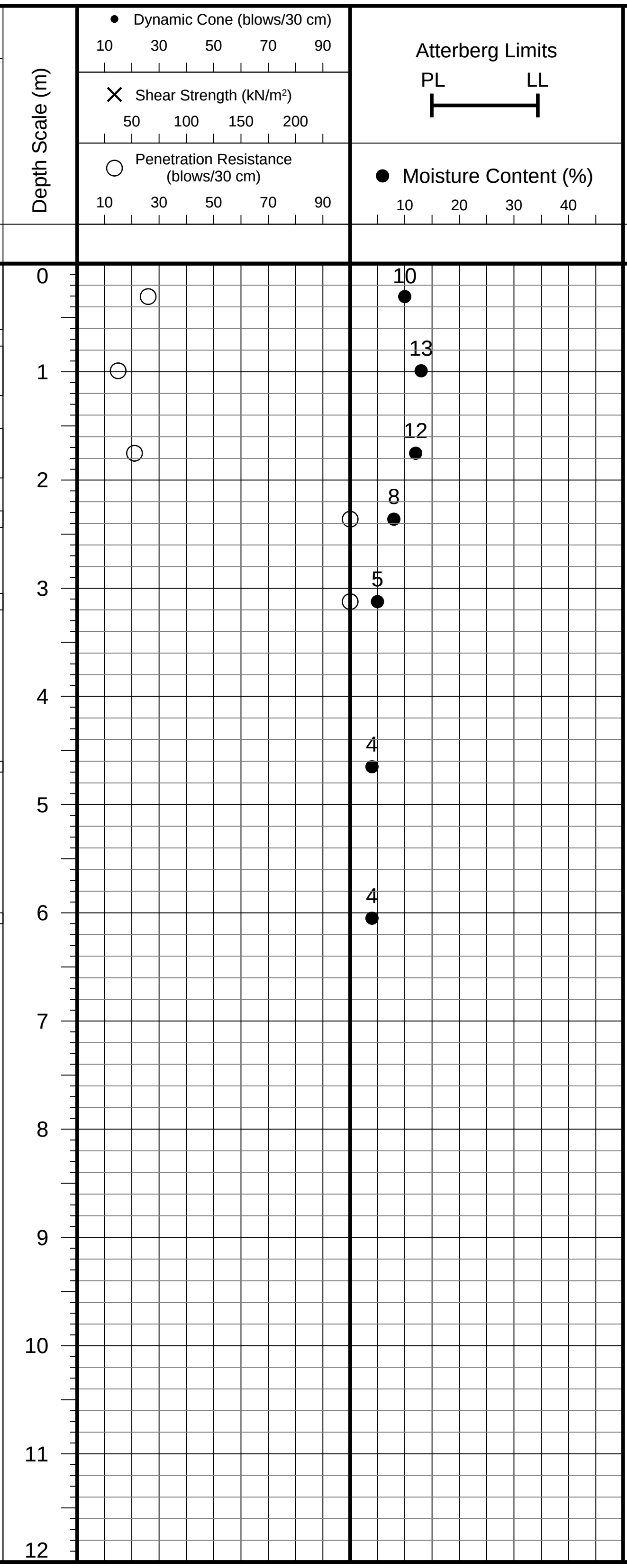
PROJECT DESCRIPTION: Proposed Mixed-Use Buildings

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2163 and 2169 Sixth Line, Town of Oakville

DRILLING DATE: October 4, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	WATER LEVEL
146.8	Pavement Surface				
0.0	**13 cm ASPHALT** **25 cm GRANULAR FILL**	1	DO	26	Dry on completion
	Reddish brown, stiff to very stiff **SILTY CLAY TILL** weathered a trace of gravel occ. sand seams and layers, cobbles and boulders	2	DO	15	
		3	DO	21	
144.5 2.3	Reddish brown, hard **SHALE** stratified, weathered occ. clay seams and limestone layers	4	DO	50/13	
		5	DO	50/5	
		6	AS	-	
		7	AS	-	
140.7 6.1	**END OF BOREHOLE**				



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JOB NO.: 1909-S038

LOG OF BOREHOLE NO.: 6

FIGURE NO.: 6

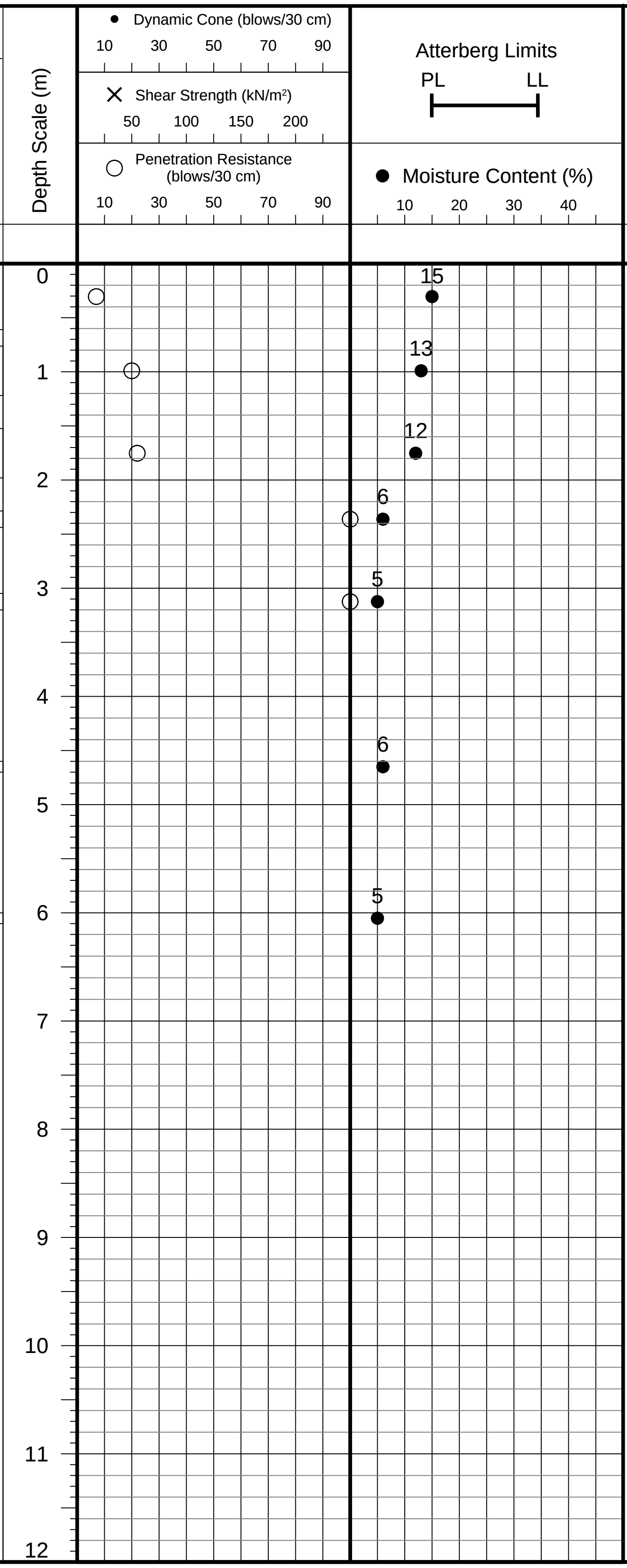
PROJECT DESCRIPTION: Proposed Mixed-Use Buildings

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2163 and 2169 Sixth Line, Town of Oakville

DRILLING DATE: October 4, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value	WATER LEVEL
148.4 0.0	Pavement Surface **13 cm TOPSOIL**				
	Reddish brown, firm to very stiff weathered **SILTY CLAY TILL** a trace of gravel occ. sand seams and layers, cobbles and boulders	1	DO	7	Dry on completion
		2	DO	20	
		3	DO	22	
146.1 2.3	Reddish brown, hard **SHALE** stratified, weathered occ. clay seams and limestone layers	4	DO	50/15	
		5	DO	50/5	
		6	AS	-	
142.3 6.1	**END OF BOREHOLE**	7	AS	-	



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JOB NO.: 1909-S038

LOG OF BOREHOLE NO.: BH/MW 7

FIGURE NO.: 7

PROJECT DESCRIPTION: Proposed Mixed-Use Buildings

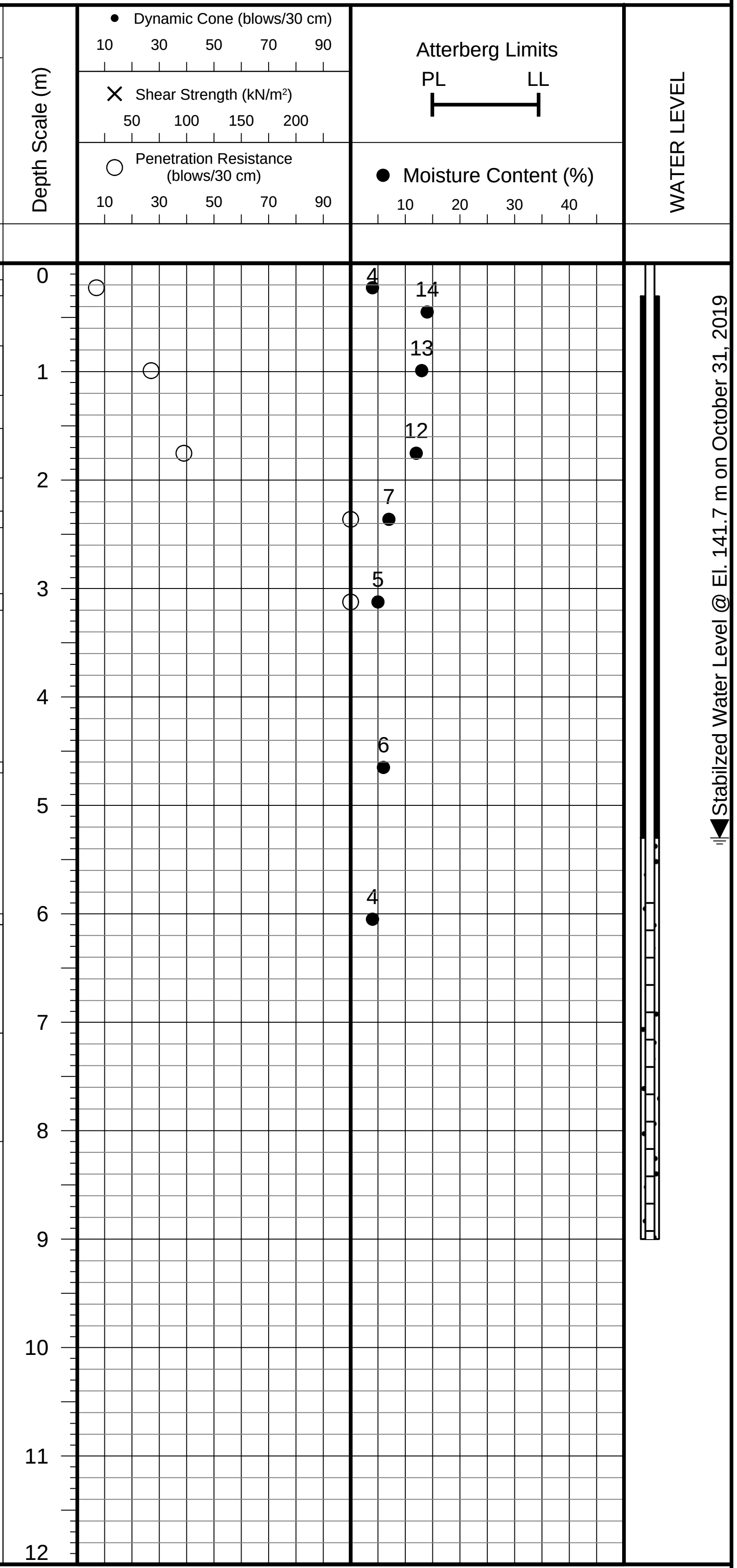
METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2163 and 2169 Sixth Line, Town of Oakville

DRILLING DATE: October 7, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
147.0	Pavement Surface			
0.0	**8 cm ASPHALT** **20 cm GRANULAR FILL**	1A 1B	DO	7
	weathered Reddish brown, very stiff to hard **SILTY CLAY TILL** a trace of gravel occ. sand seams and layers, cobbles and boulders	2	DO	27
		3	DO	39
144.7 2.3	Reddish brown, hard **SHALE** stratified weathered to 6 m	4	DO	50/15
		5	DO	50/13
		6	AS	-
		7	AS	-
		8	RC RQD	97% 73%
		9	RC RQD	100% 70%
138.9 8.1	**END OF BOREHOLE**			

Re-drilled borehole to 9 m, Installed 50 mm
Ø monitoring well to 9.0 m
3.0 m screen from 5.9 to 9.0 m
Sand backfill from 5.3 to 9.0 m
Bentonite seal from 0.3 to 5.3 m
Provided with a Flush Mount Casing



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JOB NO.: 1909-S038

LOG OF BOREHOLE NO.: BH/MW 8

FIGURE NO.: 8

PROJECT DESCRIPTION: Proposed Mixed-Use Buildings

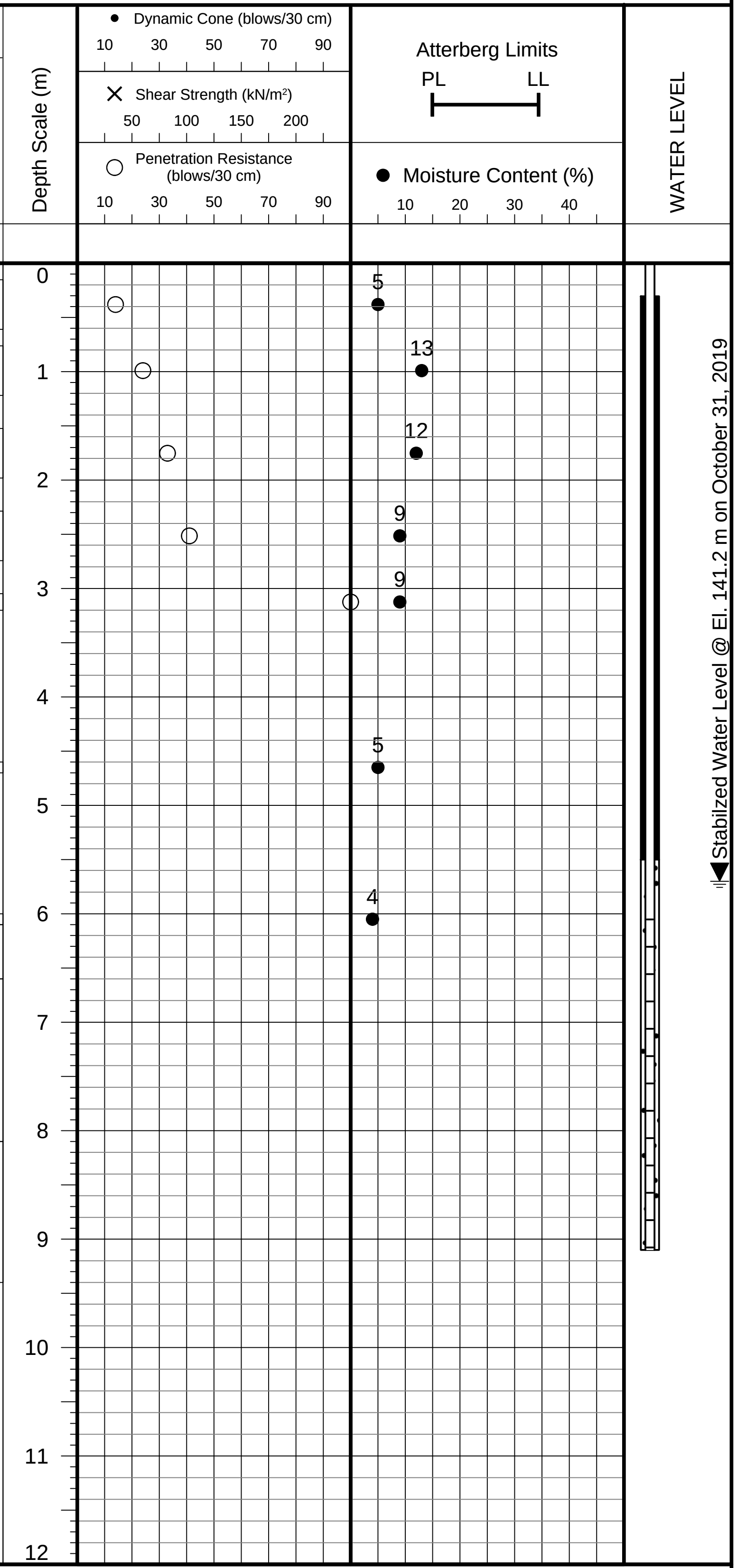
METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 2163 and 2169 Sixth Line, Town of Oakville

DRILLING DATE: October 9, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
146.9	Pavement Surface			
0.0	**8 cm ASPHALT** **52 cm GRANULAR FILL**	1	DO	14
	Reddish brown, very stiff to hard **SILTY CLAY TILL** a trace of gravel occ. sand seams and layers, cobbles and boulders	2	DO	24
		3	DO	33
		4	DO	41
143.9 3.0	Reddish brown, hard **SHALE** stratified weathered to 5.5 m occ. clay seams and limestone layers	5	DO	50/13
		6	AS	-
		7	AS	-
		8	RC RQD	100% 70%
		9	RC RQD	100% 81%
		10	RC RQD	100% 88%
137.5 9.4	**END OF BOREHOLE**			

Installed 50 mm Ø monitoring well to 9.1 m
3.0 m screen from 6.1 to 9.1 m
Sand backfill from 5.5 to 9.1 m
Bentonite seal from 0.30 to 5.5 m
Provided with a Flush Mount Casing



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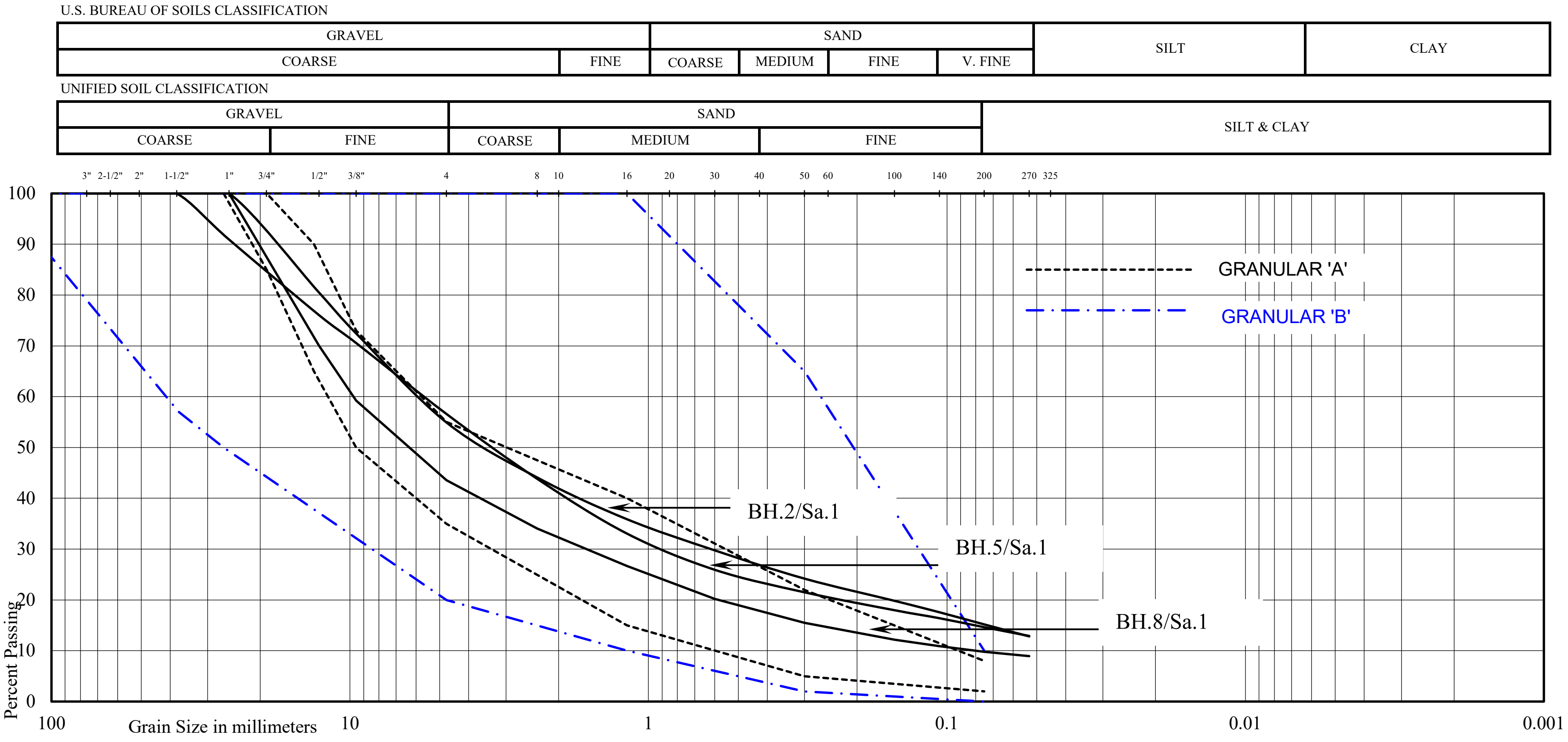
GRAIN SIZE DISTRIBUTION

Reference No: 1909-S038



Project: Mixed-Use Building

Location: 2163 and 2169 Sixth Line, Town of Okaville

Borehole No:	2	5	8
Sample No:	1	1	1
Depth (m):	0.3	0.3	0.3
Elevation (m):	148.3	146.5	146.6

BH./Sa.	2/1	5/1	8/1
Liquid Limit (%) =	-	-	-
Plastic Limit (%) =	-	-	-
Plasticity Index (%) =	-	-	-
Moisture Content (%) =	5	10	5
Estimated Permeability (cm./sec.) =	10^{-3}	10^{-3}	10^{-2}

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

Figure: 9

LEGEND

⊕ BOREHOLE

⊕ BOREHOLE and MONITORING WELL

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BOREHOLE LOCATION PLAN

SITE: 2163 and 2169 Sixth Line, Town of Oakville

DESIGNED BY: N.A

CHECKED BY: B.L

DWG NO.: 1

SCALE: 1:800

REF. NO.: 1909-S038

DATE: November 2019

REV -

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SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

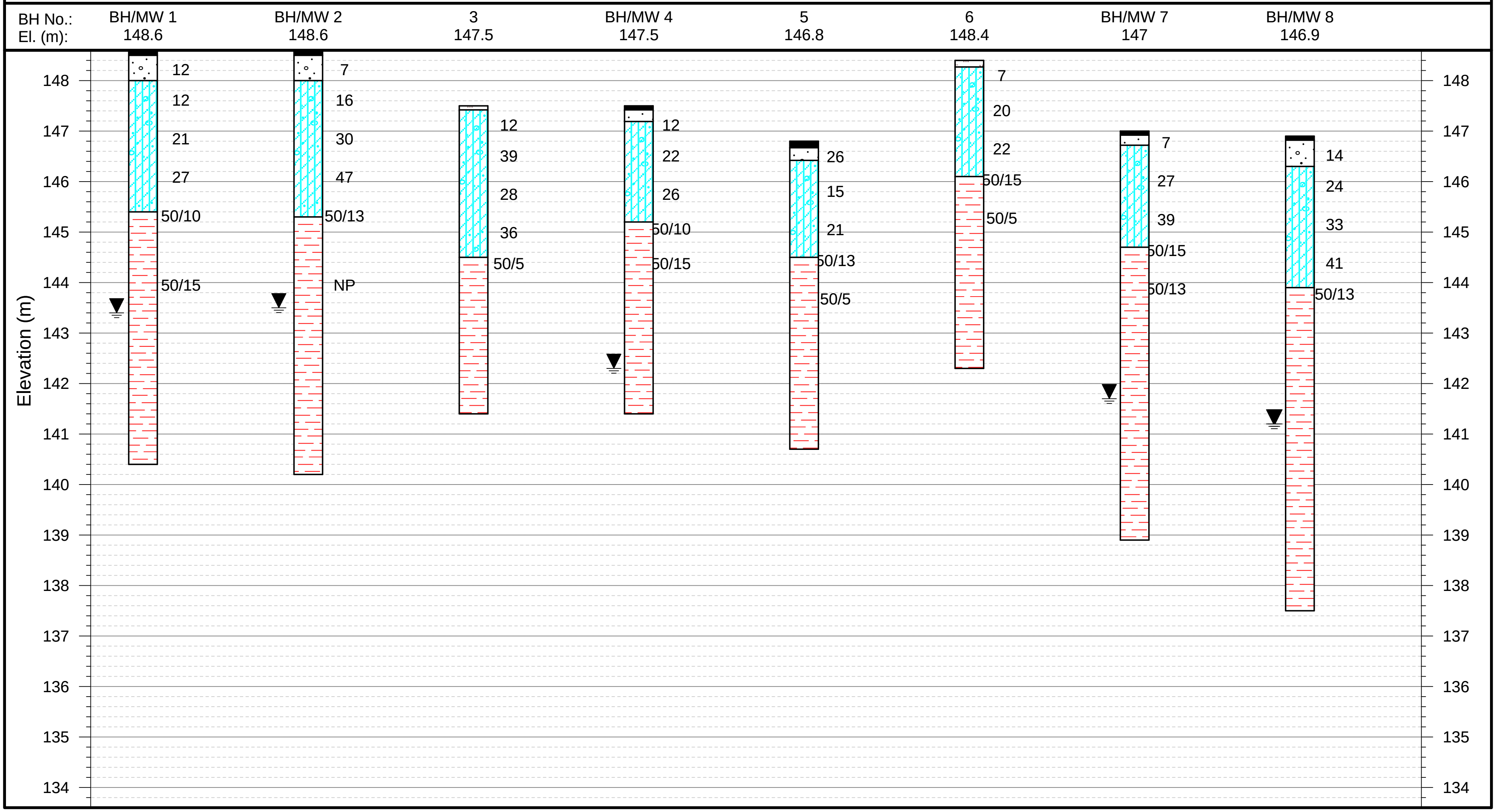
JOB NO.: 1909-S038
REPORT DATE: November 2019
PROJECT DESCRIPTION: Proposed Mixed-Use Buildings

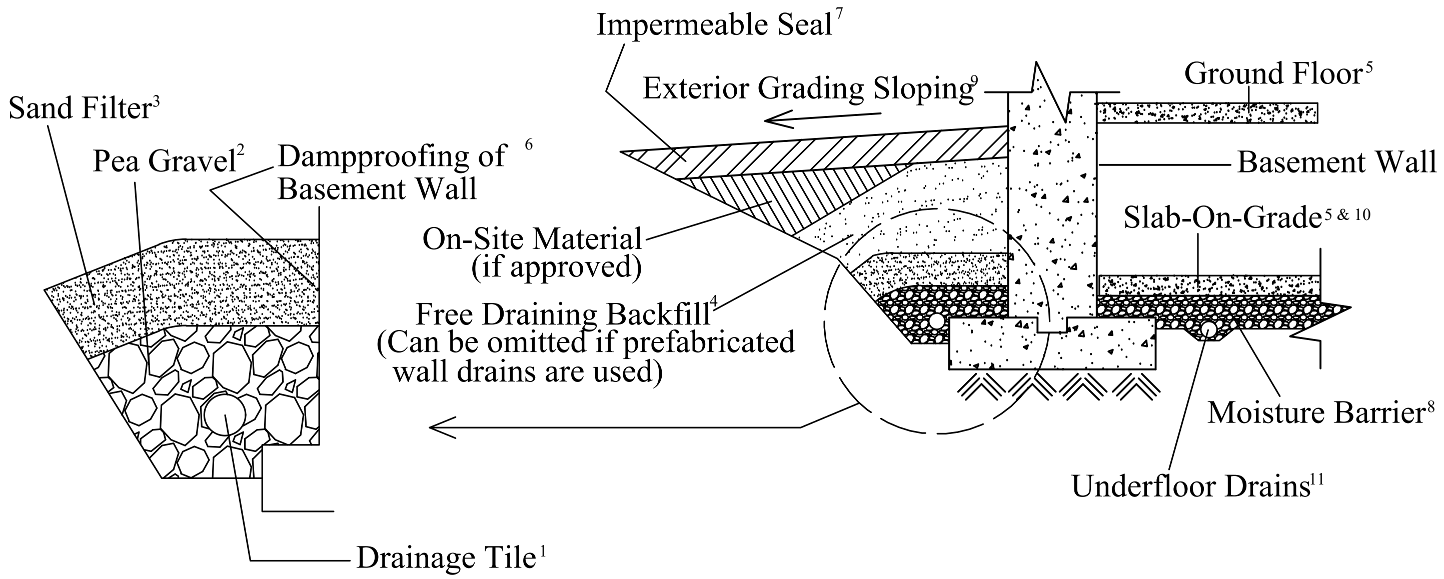
PROJECT LOCATION: 2163 and 2169 Sixth Line, Town of Oakville

LEGEND

▤ TOPSOIL ▨ GRANULAR FILL ▨ SILTY CLAY TILL ▤ SHALE
■ ASPHALT

▼ STABILIZED WATER LEVEL





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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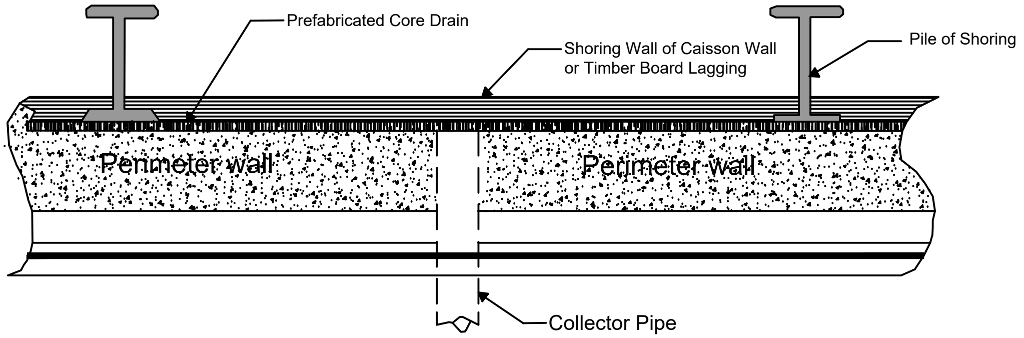
Details of Permanent Perimeter Drainage System

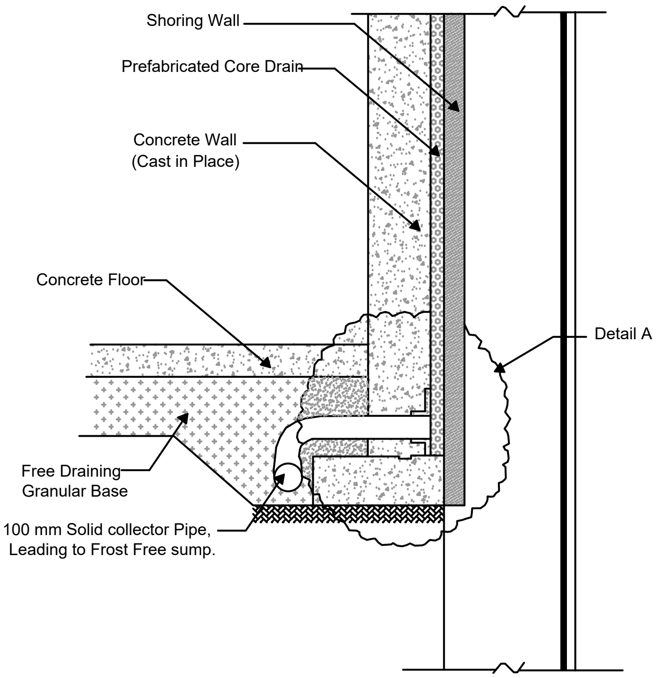
SITE 2163 and 2169 Sixth Line, Town of Oakville

DESIGNED BY N.A **CHECKED BY** B.S. **DWG NO.** 3

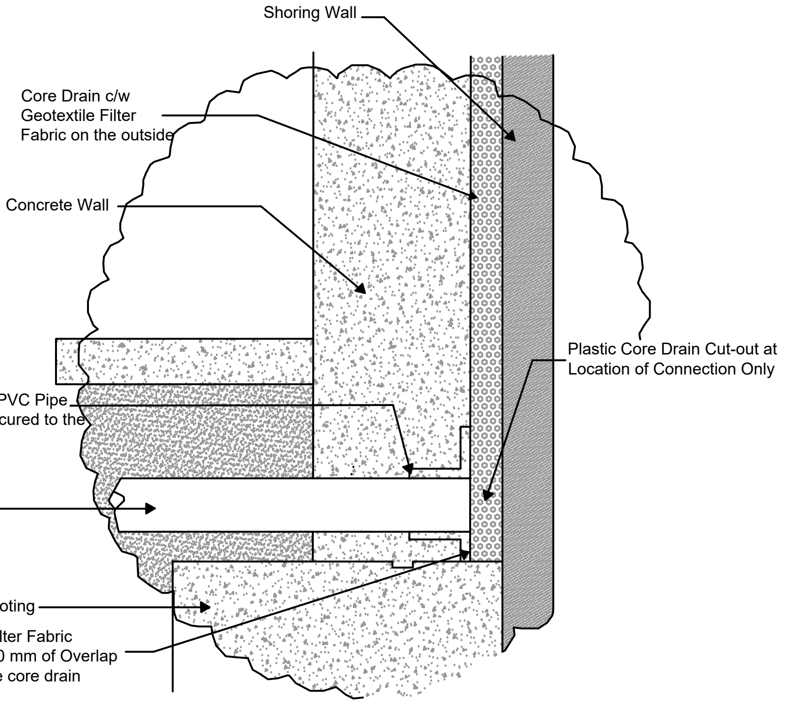
SCALE N.T.S. **REF. NO.** 1909-S038 **DATE** November 2019 **REV** -



PLAN



TYPICAL SECTION



DETAIL A

NOTES:

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

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

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

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

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Permanent Perimeter Drainage System

SITE: 2163 and 2169 Sixth Line, Town of Oakville

DESIGNED BY: N.A	**CHECKED BY:** B.L	**DWG NO.:** 4	
SCALE: N.T.S.	**REF. NO.:** 1909-S038	**DATE:** November 2019	**REV** -

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

APPENDIX

SHORING DESIGN

REFERENCE NO. 1909-S038

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:

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 subsoil for pile support
L = Embedment depth of the pile (m)
γ = Unit weight of subsoil below bottom of excavation (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 for pile support =100 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 can be estimated using an adhesion value of 60 kPa. Full scale load tests should be carried out on the tieback anchors in each type of soils and at each level of anchor support at the site to confirm the design parameters and the adhesion values. The test anchors should be loaded in a pattern as described in CFEM, to 200% of the design load or until there is a significant increase in the pullout rate. In the latter case, the design load must be limited to 50% of the maximum load at which the pullout increases. Based on the results of the pullout test, it may be necessary to modify the anchor design of the production 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 450 kPa.

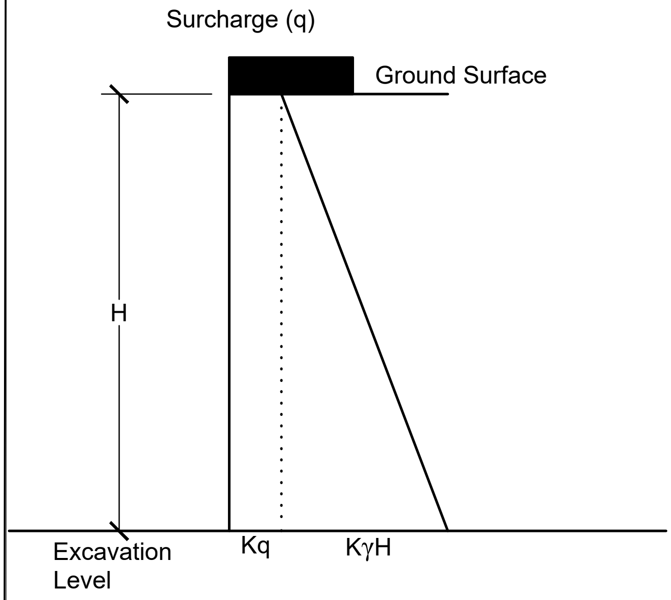
The raker footings should be located outside the zone of influence of the buried portion of the soldier piles at a distance of not less than 1.5 of the length of embedment of the soldier pile.

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

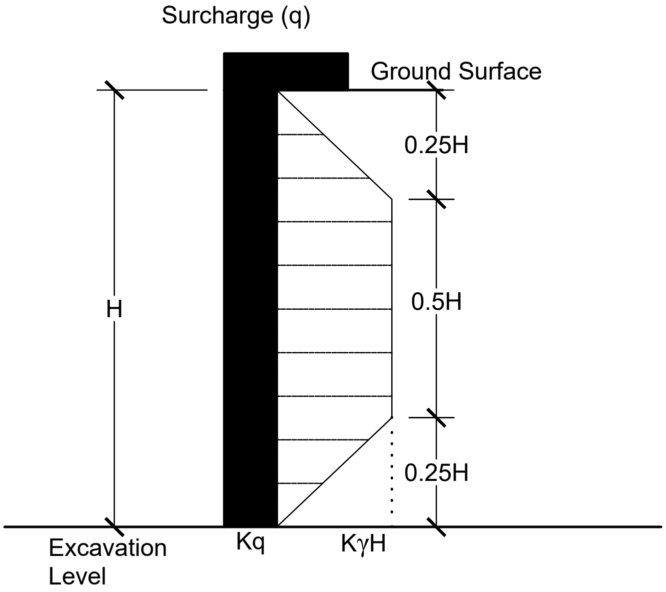
MONITORING OF PERFORMANCE

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

SITE: 2163 and 2169 Sixth Line, Town of Oakville

DESIGNED BY: N.A
CHECKED BY: B.L
DWG NO.: A1

SCALE: N.T.S.
REF. NO.: 1909-S038
DATE: November 2019
REV -