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A REPORT TO
2426009 ONTARIO LTD.

A GEOTECHNICAL INVESTIGATION FOR PROPOSED RESIDENTIAL DEVELOPMENT

6460 MAIN STREET

TOWN OF WHITCHURCH-STOUFFVILLE

REFERENCE NO. 2203-S017

AUGUST 2022

DISTRIBUTION

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1.0 INTRODUCTION

In accordance with written authorization dated February 28, 2022, from Mr. Terry Shim of 2426009 Ontario Ltd., a geotechnical investigation was carried out at 6460 Main Street in the Town of Whitchurch-Stouffville.

The purpose of this investigation was to reveal the subsurface conditions and determine the engineering properties of the disclosed soils for the design and construction of the proposed residential development. The geotechnical findings and resulting recommendations are presented in this Report.

2.0 SITE AND PROJECT DESCRIPTION

The Town of Whitchurch-Stouffville is situated on Halton till plain where drift dominates the soil stratigraphy. In places, the drift has been eroded by Peel ponding (glacial lake) and filled with glacial flow sands and lacustrine sand, silt, clay and reworked till.

The subject property is located on the north side of Main Street, approximately 55 m west of Baker Street. The site is currently a vacant field and is located in a residential area. The ground surface is relatively level with some undulations, with the highest elevation at the northeast portion of the property.

Based on the site plan prepared by Arrowsmith and Associates revised February 4, 2019, it is understood that the proposed project will consist of low-rise townhouse units with a paved area for access and parking.

3.0 FIELD WORK

The field work, consisting of three (3) boreholes to a depth of 6.6 m below the prevailing ground surface, was performed on March 16, 2022, at locations shown on the Borehole Location Plan, Drawing No.1.

The boreholes were advanced 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 intervals to the sampling depths. The test results are recorded as the Standard Penetration Resistance (or ‘N’ values) of the subsoil. The relative density of the non-cohesive strata and the consistency of the cohesive strata are inferred from the ‘N’ values. Split-spoon samples were recovered for soil classification and laboratory testing.

Upon the completion of drilling and sampling, monitoring wells were installed in all the boreholes for groundwater monitoring. Details of the monitoring wells are included in the corresponding borehole logs. The groundwater recording in the monitoring wells and the hydrogeological assessment was performed by another consultant.

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

The ground elevation of each borehole location was determined using hand-held Global Navigation Satellite System survey equipment (Trimble Geoexplorer 6000).

4.0 **SUBSURFACE CONDITIONS**

Detailed descriptions of the encountered subsurface conditions from the boreholes are presented on the Borehole Logs, comprising Figures 1 to 3, inclusive. The revealed stratigraphy is plotted on the Subsurface Profiles, Drawing No. 2. The engineering properties of the disclosed soils are discussed herein.

The investigation has disclosed that beneath a layer of topsoil, the site is generally underlain by strata of silty clay till and sandy silt till.

4.1 **Topsoil** (All Boreholes)

The topsoil, approximately 20 to 31 cm in thickness, was encountered at the ground surface at all borehole locations. It can only be reused for general landscaping purpose.

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

The silty clay till is the predominant soil type founded on site extending to the maximum investigated depth in 2 boreholes. Borehole 3 encountered the silty clay till at a depth of 1.2 to 4.6 m. Sample examinations show that the particle sizes of the till range from clay to gravel, with the clay fraction exerting the dominant influence on the soil properties. Occasional sand seams and layers, and cobbles and boulders were also detected in the clay till mantle. It is heterogeneous and amorphous in structure, indicating that it is a glacial deposit.

The obtained 'N' values ranged from 0 to 38, with a median of 21 blows per 30 cm penetration, from which the consistency of the silty clay till is inferred as very soft to hard, being generally very stiff. The soft soil is restricted to the weathered zone.

The Atterberg Limits of 1 representative sample and the moisture content of all samples were determined. The results are plotted on the Borehole Logs and summarized below:

Liquid Limit	23%
Plastic Limit	16%
Natural Water Content	12%

The above results show the clay till is a cohesive material with low plasticity. The natural water content lies below its plastic limit, confirming the consistency of the till determined by the 'N' values. The low 'N' values and high moisture content were generally obtained in the badly weathered zone, showing the clay till in this zone has been fractured by the weathering process, allowing infiltrating precipitation to wet the fissures, thus softening its consistency.

Grain size analysis was performed on 1 representative sample and the result is presented in Figure 4.

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

- High frost susceptibility and low soil adfreezing potential
- Low permeabilty, with an estimated coefficient of permeability of 10^{-7} cm/sec, a percolation time of more than 80 min/cm.
- Low water erodibility.
- A poor pavement-supportive material, with an estimated California Bearing Ration (CBR) value of 3%.
- In excavation, the clay till will be relatively stable in steep cuts. Under prolonged exposure, localized sloughing may occur.
- Moderately high corrosivity to buried metal, with an estimated electrical resistivity of 3000 ohm·cm.

4.3 **Sandy Silt Till** (Borehole 3)

The native sandy silt till was interstratified in the silty clay till in Borehole 3 and extends to the maximum investigated depth. Grain size analysis was performed on 1 representative sample and the result is presented in Figure 5.

The obtained 'N' values range from 6 to 25, with a median of 7 blows per 30 cm penetration, showing the till deposit is loose to compact, being generally loose.

The natural water content of the samples ranged from 13% to 34%, with a median of 22%, indicating moist to wet condition.

The engineering properties of the sandy silt till deposit are presented below:

- High frost susceptibility.
- Relatively low to low permeability, with an estimated coefficient of permeability of 10^{-5} to 10^{-6} cm/sec, and a percolation time of 30 to 50 min/cm.
- The shear strength is primarily derived from internal friction, and is augmented by cementation.
- A fair pavement-supportive material, with an estimated CBR value of 8%.
- The till will be stable in relatively steep excavation; however, localized sheet collapse may occur under prolonged exposure.
- Moderate corrosivity to buried metal, with an estimated electrical resistivity of 4500 ohm·cm.

4.4 Compaction Characteristics of the Revealed Soils

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

Table 1 - Estimated Water Content for Compaction

Soil Type	Determined Natural Water Content (%)	Water Content (%) for Standard Proctor Compaction	
		100% (optimum)	**Range for 95% or +**
Silty Clay Till	7 to 56 (median 14)	16	14 to 18
Sandy Silt Till	13 to 34 (median 22)	12	8 to 16

Portions of the on-site inorganic soils are too wet to be reused for structural backfill. The wet soils should be spread thinly on the ground to allow aeration in warm and dry weather prior to reusing for structural backfill. The backfill material should be free of organics or oversized (over 15 cm) boulders. The lift thickness for compaction should be limited to no more than 20 cm thick.

5.0 GROUNDWATER CONDITIONS

The boreholes were checked for the presence of groundwater and signs of cave-in upon completion of drilling. Groundwater was contacted at a depth of 2.4 to 3.6 m, or El. 263.7 to 264.0 m; while caved-in was contacted at a depth of 3.7 to 5.5 m, or El. 261.2 to 263.0 m. The recorded groundwater level in the boreholes is summarized in Table 2.

Table 2 - Groundwater Levels Upon Completion

Borehole No.	Borehole Depth (m)	Ground Elevation (m)	Groundwater/Cave-In* Level Upon Completion	
			Depth (m)	El. (m)
1	6.6	267.6	3.6/4.6*	264.0/263.0*
2	6.6	266.7	3.0/5.5*	263.7/261.2*
3	6.6	266.4	2.4/3.7*	264.0/262.7*

Groundwater was recorded in the monitoring wells by another consultant between March 17 and June 15, 2022. Groundwater was recorded in the monitoring wells at a depth of -0.1 m to 1.7 m, or between El. 264.6 to 267.7 m.

The groundwater monitoring data between March 17 and June 15, 2022 are presented in the Appendix.

Based on the natural water content, soil stratigraphy, and water levels, perched water exists in the sand and silt layers within the till deposits and are subject to seasonal fluctuation. Continuous groundwater, however, is not anticipated within the depth of investigation.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation has disclosed that beneath a layer of topsoil, the site is generally underlain by strata of silty clay till and sandy silt till.

Based on the natural water content, soil stratigraphy, and water levels, perched water exists in the sand and silt layers within the till deposits and are subject to seasonal fluctuation. Continuous groundwater, however, is not anticipated within the depth of investigation.

The geotechnical findings which warrant special consideration are presented below:

1. The topsoil must be removed for site development. The topsoil can be re-used for landscaping only. Any surplus should be removed off-site.
2. In areas where the site will be regraded with additional fill, it is economical to place an engineered fill for foundation, site services and road construction.
3. The proposed structures can be supported on conventional footings, founded on engineered fill or sound native soil. The footing subgrade must be inspected by a geotechnical engineer, or a senior geotechnical technician, to ensure that the revealed conditions are compatible with the design of foundations.
4. Class 'B' bedding, consisting of compacted 19-mm Crusher-Run Limestone (CRL), or equivalent, is recommended for the construction of the underground services. The service pipes must consist of leak-proof joints, or the joints must be wrapped with a waterproof membrane.

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

6.1 Site Preparation

In areas where the site will be regraded with additional fill, it is economical to place an engineered fill for foundation, site services and road construction. The engineering requirements for a certifiable fill for road construction, municipal services and conventional footings are presented below:

1. The topsoil must be removed. The subgrade must be inspected and proof-rolled prior to any fill placement.
2. Inorganic soils must be used, and they must be uniformly compacted in 20 cm thick lifts to at least 98% Standard Proctor dry density (SPDD) up to the proposed finished grade. The soil moisture must be properly controlled near the optimum. If the foundations are to be built soon after the fill placement, the densification process for the engineered fill must be increased to 100% SPDD.
3. If the engineered fill is compacted with the moisture content on the wet side of the optimum, the underground services and pavement construction should not begin until the pore pressure within the fill mantle has completely dissipated. This must be further assessed at the time of the engineered fill construction.

4. If imported fill is to be used, it should be inorganic soils, free of deleterious or any material with environmental issue (contamination). Any potential imported earth fill from off site must be reviewed for geotechnical and environmental quality by the appropriate personnel as authorized by the developer or agency, before it is hauled to the site.
5. The engineered fill must not be placed during the period where freezing ambient temperatures occur either persistently or intermittently. This is to ensure that the fill is free of frozen soils, ice and snow. If the engineered fill is to be left over the winter months, adequate earth cover, or equivalent, must be provided for protection against frost action.
6. The fill operation must be fully supervised and monitored by a technician under the direction of a geotechnical engineer.
7. The engineered fill envelope and finished elevations must be clearly and accurately defined in the field, and they must be precisely documented.
8. Any excavation carried out in certified engineered fill must be reported to the geotechnical consultant who supervised the fill placement in order to document the locations of the excavation and/or to supervise reinstatement of the excavated areas to engineered fill status. If construction on the engineered fill does not commence within a period of 2 years from the date of certification, the condition of the engineered fill must be assessed for re-certification.
9. Foundations founded on engineered fill must be reinforced in the footings and in the upper section of the foundation walls. It should be designed by a structural engineer to allow distribution of stress induced by the abrupt differential settlement (about 20 mm) in engineered fill.
10. The footing and underground services subgrade must be inspected by the geotechnical consulting firm that supervised the engineered fill placement. This is to ensure that the foundations and service pipes are placed within the engineered fill envelope, and the integrity of the fill has not been compromised by interim construction, environmental degradation and/or disturbance by the footing excavation.
11. In sewer construction, the engineered fill is considered to have the same structural proficiency as a native inorganic soil.

6.2 Foundations

The proposed low rise townhouse units can be constructed on conventional footings founded on the undisturbed native soil. The recommended bearing pressures for conventional footing design are presented below:

- Maximum Soil Bearing Pressure at Serviceability Limit State (SLS) = 150 kPa
- Factored Ultimate Bearing Pressure at Ultimate Limit State (ULS) = 220 kPa

Weak sandy silt till was encountered between depths of N4.0 m and 6.1 m in the vicinity of Borehole 3 located in the southeast portion of the property. Lower soil bearing pressures of 75 kPa (SLS) and 120 kPa (ULS) are recommended for the design of spread and strip footings in this area. Alternatively, the foundation can extend to competent ground at a depth below 6.1 m, or below El. 260.3 m, and design with the recommended soil bearing pressures of 150 kPa (SLS) and 220 kPa (ULS).

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

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

Where water seepage is encountered during footing excavations, or where the subgrade of the foundations is found to be wet, the subgrade should be protected by a concrete mud-slab immediately after exposure. This will prevent construction disturbance and costly rectification.

Footings exposed to weathering or in unheated areas, should have at least 1.2 m of earth cover for protection against frost action or must be adequately insulated.

Where stepped footings are required, a gradient of 10 horizontal: 7vertical must be maintained.

The proposed structures must be designed in accordance with the latest Ontario Building Code. The proposed structures should be designed to resist an earthquake force using Site Classification ‘D’ (stiff soil).

6.3 **Basement Structure**

In conventional design, perimeter subdrains and damp-proofing of the foundation walls will be required, as shown on Drawing No. 3. The subdrains should be encased in a fabric filter to protect them against blockage by silting.

The basement walls should be designed to sustain a lateral earth pressure calculated using the soil parameters stated in Section 6.8. Any applicable surcharge loads beside the basement must also be included in the design of underground structure.

The subgrade must consist of sound native soils or properly compacted inorganic fill. Any weak or wet soil should be subexcavated, sorted free of any deleterious material, aerated and uniformly compacted to at least 98% SPDD. The final subgrade must be inspected and assessed by proof-rolling prior to placement of granular bedding.

The basement floor slab should be constructed on a granular bedding, at least 20 cm in thickness, consisting of 19-mm CRL, or equivalent, compacted to 100% SPDD.

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

6.4 **Underground Services**

The subgrade for underground services should consist of properly compacted inorganic earth fill or sound native soils. Where weak or wet subgrade is encountered, it can be further subexcavated to competent ground and replaced with bedding material compacted to 98% SPDD in lifts no more than 20 cm in thickness. A Class 'B' bedding, consisting of compacted 19-mm CRL or equivalent, is recommended for the design of the underground services construction.

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

The pipe joints connecting into manholes and catch basins should be leak-proof or wrapped with a waterproof membrane. Openings to subdrains 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 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 pipe alignment at the time of services construction and must meet the minimum requirement as specified by the Town.

6.5 Backfilling in Trenches and Excavated Areas

Portions of the on site inorganic soils are too wet for use as trench backfill. The wet soils should be spread thinly on the ground to allow aeration in warm and dry weather prior to reusing as trench backfill. Where imported soil is used for backfilling, it must be free of organics and free of contamination.

The backfill in service trenches or beside foundation walls should be compacted to at least 95% SPDD in 20 cm thick layers, or the lift thickness should be determined by test strips. Below floor areas or in the zone within 1.0 m below the pavement subgrade, the material should be compacted to 98% SPDD with the water content 2% to 3% drier than the optimum.

In normal construction practice, the problem areas of pavement settlement largely occur adjacent to foundation walls, manholes, catch basins and services crossings. In areas which are inaccessible to a heavy compactor, granular backfill should be used in order to achieve the compaction with a light equipment. The requirement of this can be assessed at the time of construction.

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

- When construction is carried out in freezing winter weather, allowance should be made for these following conditions. Despite stringent backfill monitoring, frozen soil layers may inadvertently be mixed with the structural trench backfill. Should the in-situ soils have a water content on the dry side of the optimum, it would be impossible to wet the soils due to the freezing condition, rendering difficulties in obtaining uniform and proper compaction. Furthermore, the freezing condition will prevent wetting of the backfill when it is required, such as in a narrow vertical trench section, or when the trench box is removed. The above will invariably cause backfill settlement that may become evident within 1 to several years, depending on the depth of the trench which has been backfilled.
- In areas where the construction is carried out during the winter months, prolonged exposure of the trench walls will result in frost heave within the soil mantle of the walls. This may result in some settlement as the frost recedes, and repair costs will be incurred prior to final surfacing of the new pavement and the slab-on-grade construction.
- In deep trench backfill, one must be aware that future settlement may occur, unless the side of the cut is flattened to at least 2H:1V, and the lifts of the fill and its moisture

content are stringently controlled; i.e., lifts should be no more than 20 cm (or less if the backfilling conditions dictate) and uniformly compacted to achieve at least 98% SPDD, with the moisture content controlled near the optimum.

- It is often difficult to achieve uniform compaction of the backfill in the lower vertical section of a trench which is stabilized by a trench box. These sectors must be backfilled with sand or non shrinkable fill, and the compaction must be carried out diligently prior to the placement of the backfill above this sector; i.e., in the upper sloped trench section. This measure is necessary in order to prevent consolidation of inadvertent voids and loose backfill which will compromise the compaction of the backfill in the upper section.
- In areas where groundwater movement is expected in the trench backfill, anti-seepage collars (OPSS 802.095) should be provided.

6.6 Garages, Driveways, Sidewalks, Interlocking Stone Pavement

Due to the high frost susceptibility of the underlying soils, movement of the pavement structure and sidewalk can be expected during the cold seasons. In areas where ground movement cannot be tolerated, the subgrade should consist of free-draining, non-frost-susceptible granular material such as Granular ‘B’. The material must extend to a depth of 0.3 to 1.2 m, depending on the degree of tolerance for movement, and be provided with positive drainage, such as weeper subdrains connected to manholes or catch basins to minimize the movement by preventing the accumulation of water in the granular base.

The driveway leading to the garage should be backfilled with non-frost susceptible granular material with a frost taper at a slope of 1H:1V or gentler. The subgrade of the garage floor and the interior garage foundation walls should be insulated with 50-mm Styrofoam, or its thermal equivalent.

The ground surface must be graded to direct water away from the structures to minimize the frost heave phenomenon generally associated with the disclosed soil.

6.7 Pavement Design

The recommended pavement design for the proposed driveway and parking area is presented in Table 3.

Table 3 - Road Pavement Design

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	50	HL-8
Granular Base	150	19-mm CRL or equivalent
Granular Sub-base	350	50-mm CRL or equivalent

In preparation of the pavement subgrade, the subgrade surface must be proof-rolled using a heavy roller or loaded dump truck. Any soft spot identified must be subexcavated, and replaced with dry inorganic material and properly compacted in layers.

In the zone within 1.0 m below the pavement subgrade, the backfill should be compacted to at least 98% SPDD, with the water content 2% to 3% drier than the optimum in 20 cm layers, or the lift thickness should be determined by test strips. In the lower zone, a 95% SPDD compaction is adequate.

All the granular bases should be compacted to 100% SPDD.

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

- The lot areas adjacent to the road pavement should be properly graded to prevent ponding of water.
- The pavement subgrade should be properly crowned and smooth-rolled to allow interim precipitation to be properly drained.
- Fabric filter-encased curb subdrains on both sides of the roadway are required to meet the Town’s requirements.
- If the pavement is to be constructed during the wet seasons and extremely soft subgrade occurs, the granular sub-base may require thickening. This can be further assessed during construction.

6.8 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 (kN/m^3)**		**Estimated Bulk Factor**	
	Bulk	**Submerged**	**Loose**	**Compacted**
Weathered Tills	21.5	11.0	1.25	1.00
Sound Tills	22.0	11.5	1.33	1.05
Lateral Earth Pressure Coefficients		**Active K_a**	**At Rest K_0**	**Passive K_p**
Silty Clay Till		0.35	0.45	2.86
Sandy Silt Till		0.30	0.40	3.33

6.9 Excavation

All 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
Sound Tills	2
Weathered Tills	3

The groundwater yield from the sand and silt seams and layers in the till deposits is expected to be small and limited. Where groundwater seepage is encountered in an open excavation, the groundwater is expected to be controllable by pumping from sumps. However, this should be verified with the hydrogeological report, by others.

Prospective contractors must be asked to assess the in situ subsurface conditions for soil cuts by digging test pits. These test pits should be allowed to remain open for a few hours to assess the stability and trenching conditions.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of 2426009 Ontario Ltd. and for review by the 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 Thomas Tingson, EIT. and Bernard Lee, P.Eng., in light of the information available to it at the time of preparation.

Any use which a Third Party makes of this report, or any reliance on decisions to be made based on it, 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.

SOIL ENGINEERS LTD.

Ag 12, 2022
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LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

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

SAMPLE TYPES

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

PENETRATION RESISTANCE

Dynamic Cone Penetration Resistance:

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

Plotted as '—●—'

Standard Penetration Resistance or 'N' Value:

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

Plotted as '○'

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

SOIL DESCRIPTION

Cohesionless Soils:

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

Cohesive Soils:

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

Method of Determination of Undrained Shear Strength of Cohesive Soils:

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

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

METRIC CONVERSION FACTORS

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

JOB NO.: 2203-S017

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

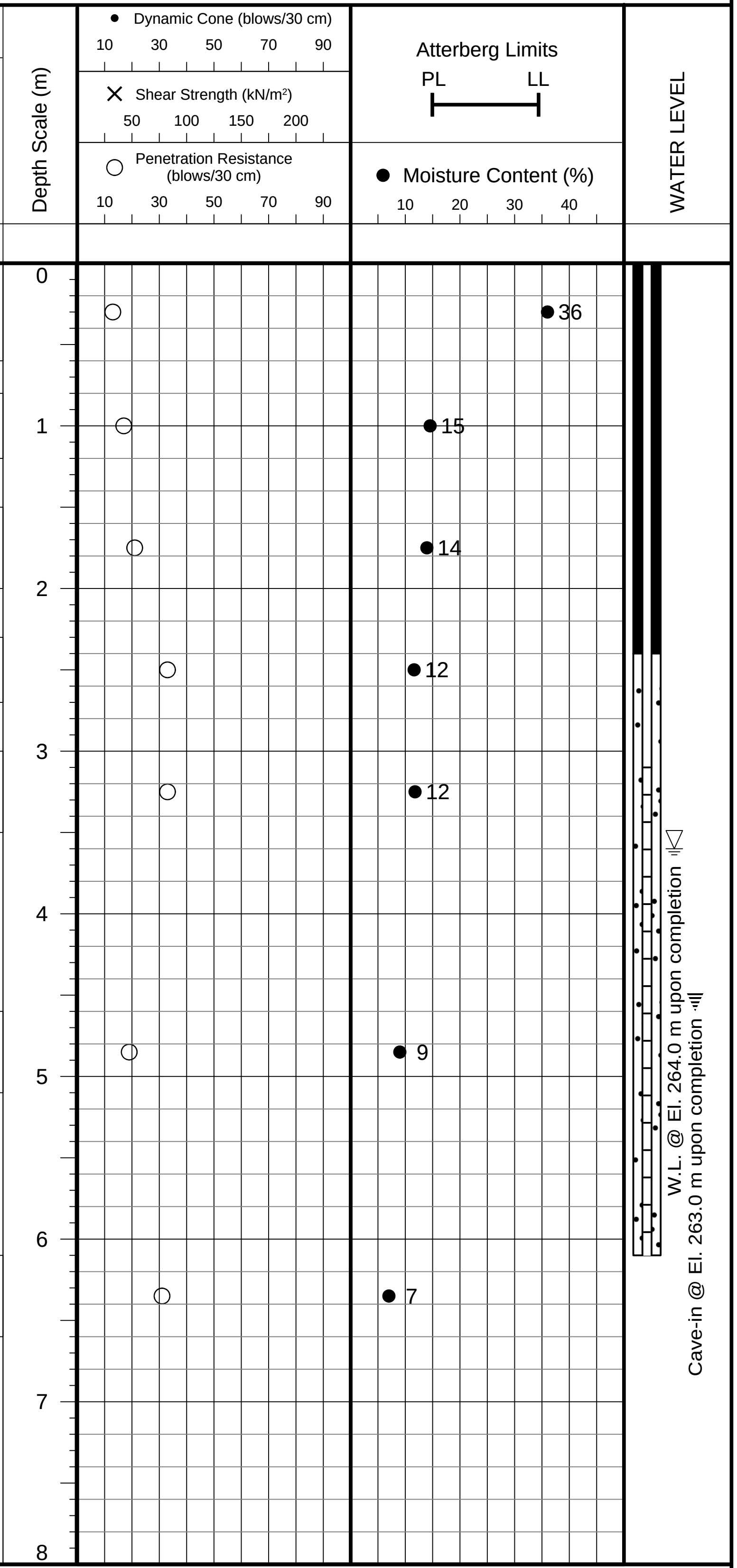
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight Auger (Solid Stem)

PROJECT LOCATION: 6460 Main Street, Town of Whitchurch-Stoufville

DRILLING DATE: March 16, 2022

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES		
		Number	Type	N-Value
267.6	Ground Surface			
0.0	**20 cm TOPSOIL**			
	Stiff to hard **SILTY CLAY TILL** a trace of gravel occ. sand seams and layers, cobbles and boulders	1	DO	13
		2	DO	17
		3	DO	21
		4	DO	33
	— — brown grey	5	DO	33
		6	DO	19
		7	DO	31
261.0 6.6	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 6.1 m completed with 3.1 m screen Sand backfill from 2.4 to 6.1 m Bentonite seal from 0.0 m to 2.4 m Provided with a monument steel casing			



Soil Engineers Ltd.

JOB NO.: 2203-S017

LOG OF BOREHOLE NO.: 2

FIGURE NO.: 2

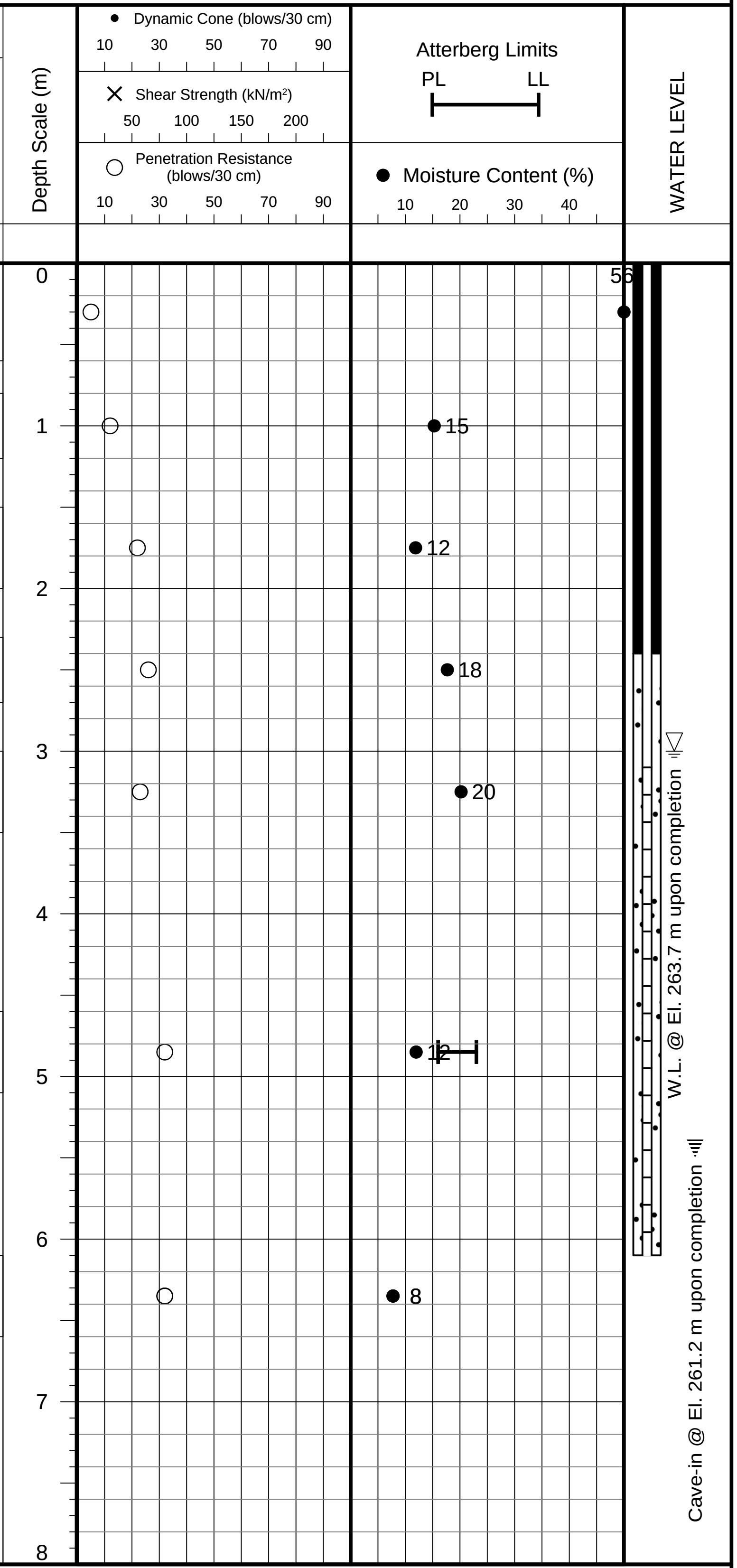
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight Auger
(Solid Stem)

PROJECT LOCATION: 6460 Main Street, Town of Whitchurch-Stoufville

DRILLING DATE: March 16, 2022

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
266.7 0.0	Ground Surface **20 cm TOPSOIL**			
	Very soft to hard **SILTY CLAY TILL** a trace of gravel occ. sand seams and layers, cobbles and boulders weathered	1	DO	0
		2	DO	11
		3	DO	21
		4	DO	20
	brown grey silt layer	5	DO	22
	sand layer	6	DO	33
		7	DO	35
260.1 6.6	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 6.1 m completed with 3.1 m screen Sand backfill from 2.4 to 6.1 m Bentonite seal from 0.0 m to 2.4 m Provided with a monument steel casing			



Soil Engineers Ltd.

JOB NO.: 2203-S017

LOG OF BOREHOLE NO.: 3

FIGURE NO.: 3

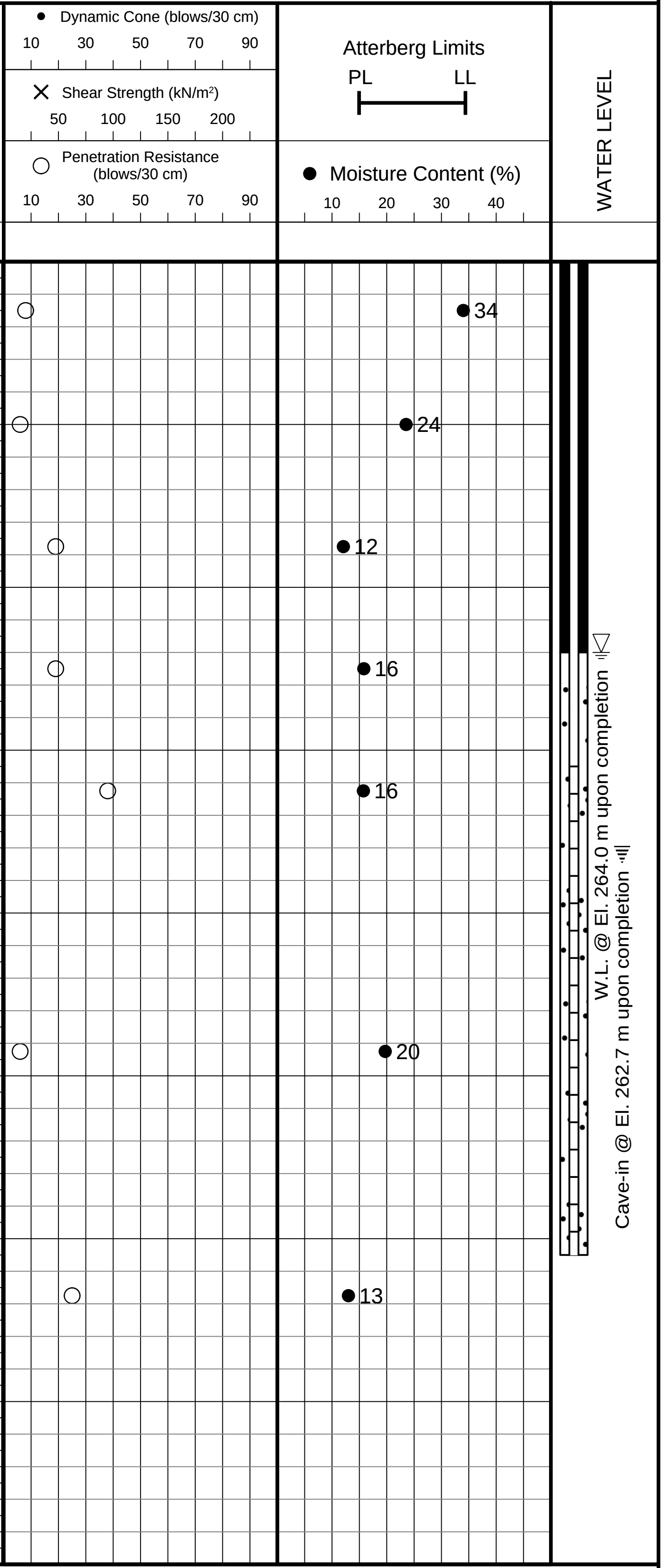
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight Auger (Solid Stem)

PROJECT LOCATION: 6460 Main Street, Town of Whitchurch-Stoufville

DRILLING DATE: March 16, 2022

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)
266.4	Ground Surface				
0.0	**31 cm TOPSOIL**				0
	Brown, loose **SANDY SILT TILL** some clay	1	DO	8	
		2	DO	6	1
265.2 1.2	Brown, very firm to hard **SILTY CLAY TILL** sandy, a trace of gravel occ. cobbles and boulders	3	DO	19	2
	silt layer	4	DO	19	
		5	DO	38	3
					4
261.8 4.6	Grey, loose to compact **SANDY SILT TILL** some clay	6	DO	6	5
	sand layer				6
		7	DO	25	
259.8 6.6	**END OF BOREHOLE** Installed 50 mm Ø monitoring well to 6.1 m completed with 3.1 m screen Sand backfill from 2.4 to 6.1 m Bentonite seal from 0.0 m to 2.4 m Provided with a monument steel casing				7
					8



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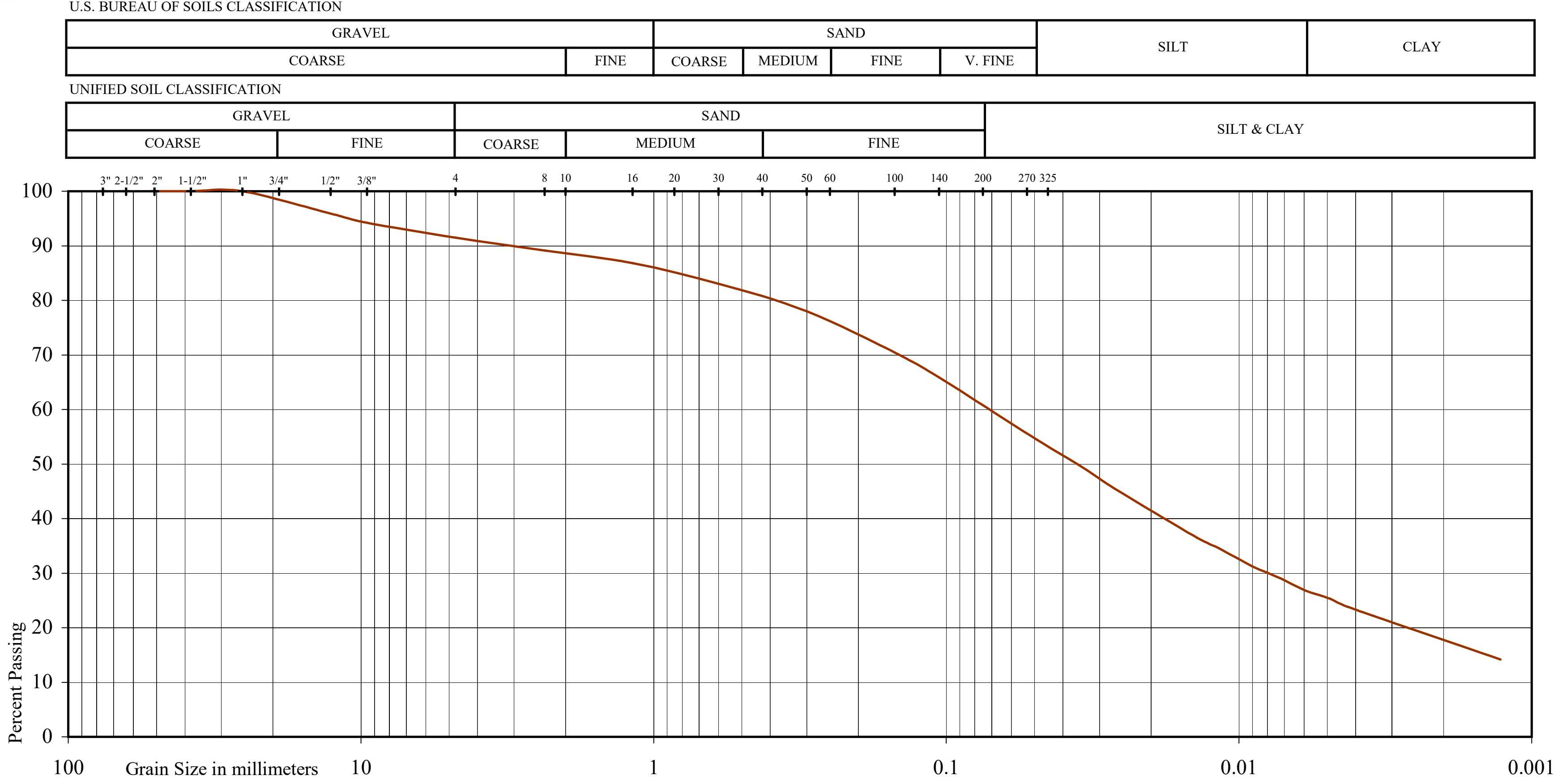
GRAIN SIZE DISTRIBUTION

Reference No: 2203-S017



Project: Proposed Residential Development

Location: 6460 Main Street, Town of Whitchurch-Stoufville

Borehole No: 2

Sample No: 6

Depth (m): 4.8

Elevation (m): 261.9

Liquid Limit (%) = 23

Plastic Limit (%) = 16

Plasticity Index (%) = 7

Moisture Content (%) = 12

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

Classification of Sample [& Group Symbol]: SILTY CLAY TILL
sandy, a trace of gravel

Figure: 4

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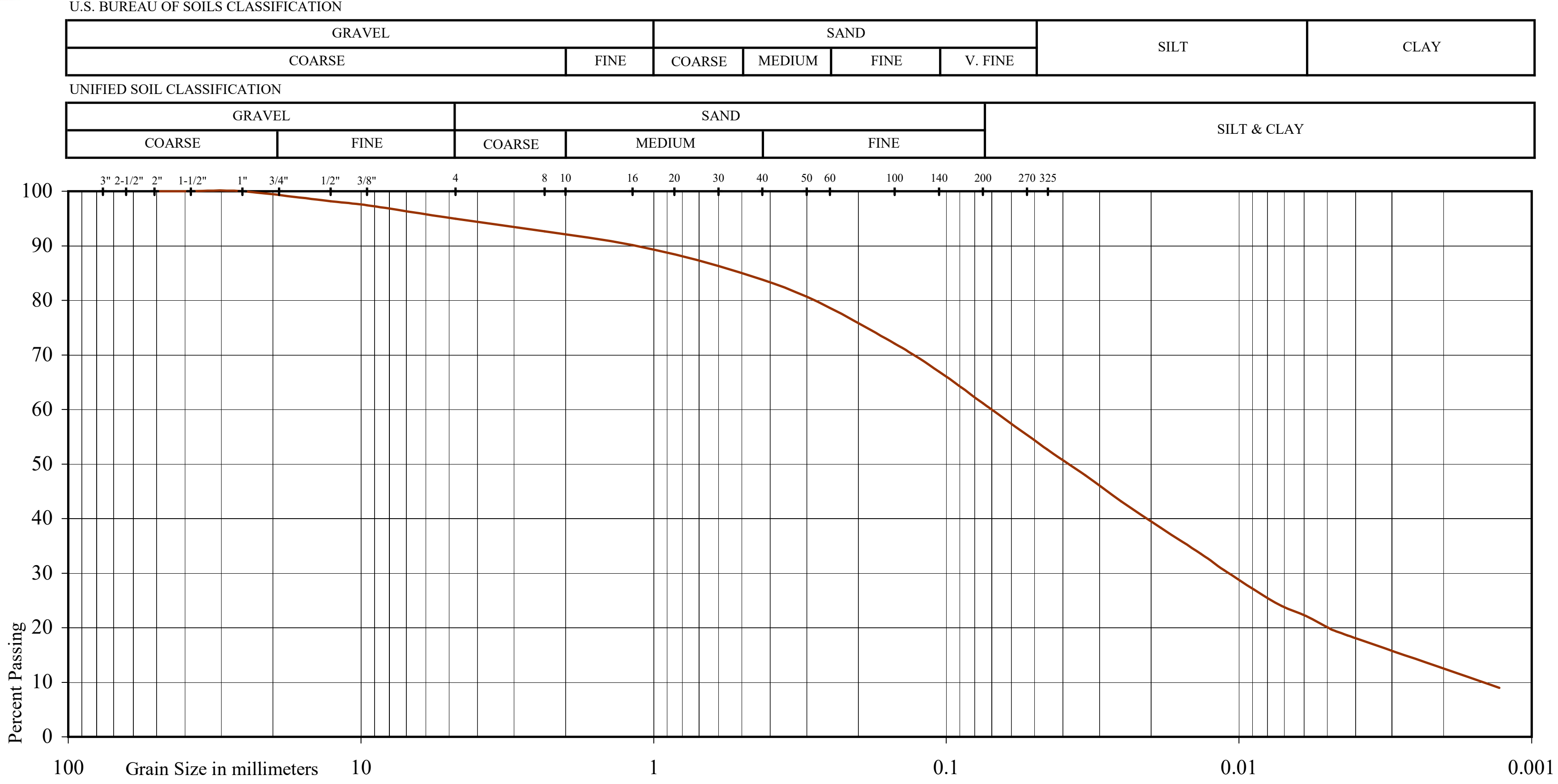
GRAIN SIZE DISTRIBUTION

Reference No: 2203-S017



Project: Proposed Residential Development

Location: 6460 Main Street, Town of Whitchurch-Stoufville

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 24

Estimated Permeability (cm./sec.) = 10^{-6}

Borehole No: 3

Sample No: 2

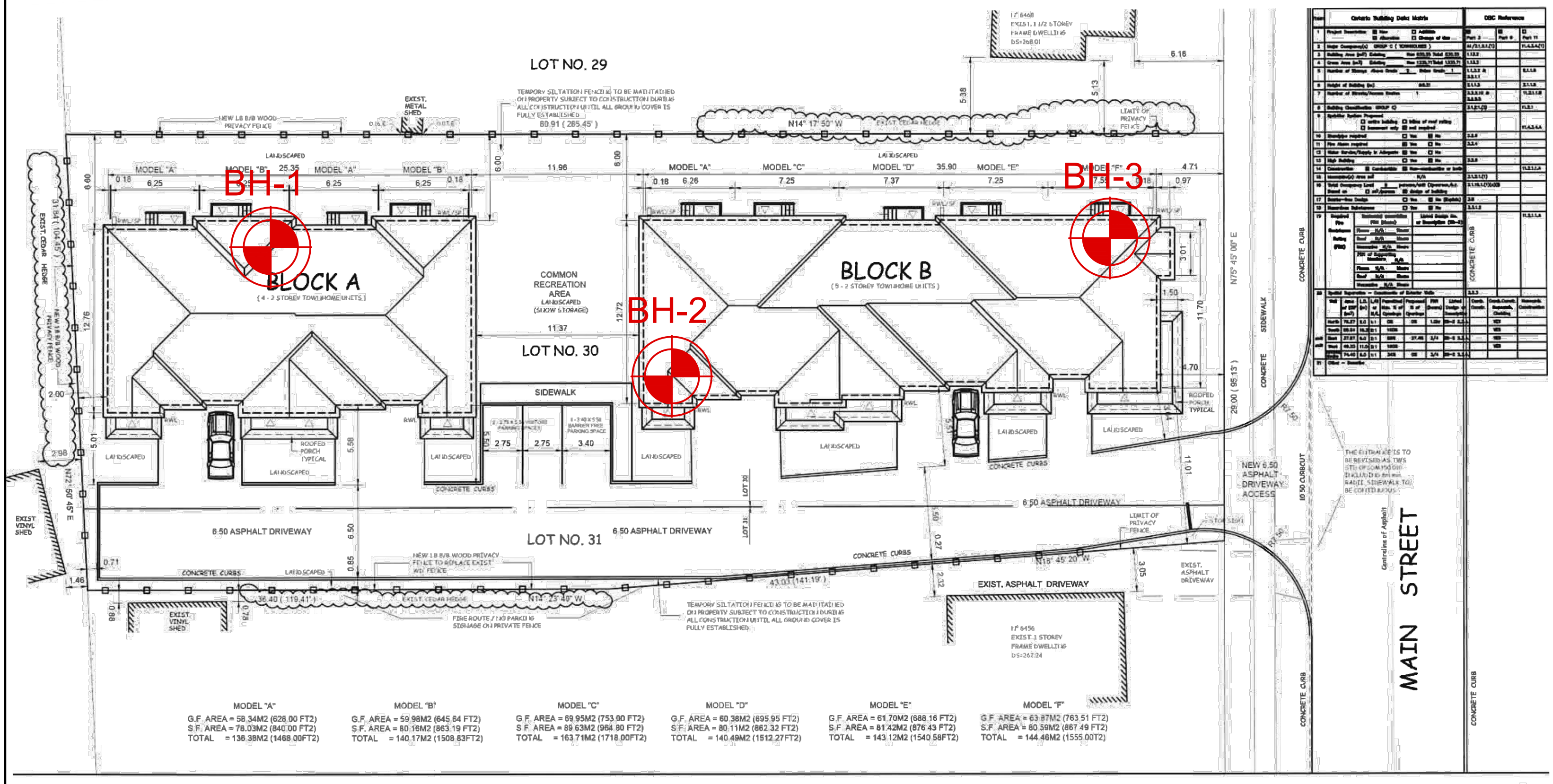
Depth (m): 1

Elevation (m): 265.4

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

Figure: 5

Key Plan N.T.S.



MODEL "A"
G.F. AREA = 58.34M2 (628.00 FT2)
S.F. AREA = 78.03M2 (840.00 FT2)
TOTAL = 136.38M2 (1468.00FT2)

MODEL "B"
G.F. AREA = 59.98M2 (645.64 FT2)
S.F. AREA = 80.16M2 (863.19 FT2)
TOTAL = 140.17M2 (1508.83FT2)

MODEL "C"
G.F. AREA = 69.95M2 (753.00 FT2)
S.F. AREA = 89.63M2 (964.80 FT2)
TOTAL = 163.71M2 (1718.00FT2)

MODEL "D"
G.F. AREA = 60.38M2 (695.95 FT2)
S.F. AREA = 80.11M2 (862.32 FT2)
TOTAL = 140.49M2 (1512.27FT2)

MODEL "E"
G.F. AREA = 61.70M2 (688.16 FT2)
S.F. AREA = 81.42M2 (876.43 FT2)
TOTAL = 143.12M2 (1540.58FT2)

MODEL "F"
G.F. AREA = 63.87M2 (763.51 FT2)
S.F. AREA = 80.59M2 (867.49 FT2)
TOTAL = 144.46M2 (1555.00T2)

LEGEND

⊕ Borehole with Monitoring Wells

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Borehole Location Plan

SITE: 6460 Main Street, Town of Whitchurch-Stouffville

DESIGNED BY: T.T.
CHECKED BY: B.L.
DWG NO.: 1
SCALE: 1:500
REF. NO.: 2203-S017
DATE: August 2022
REV

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SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

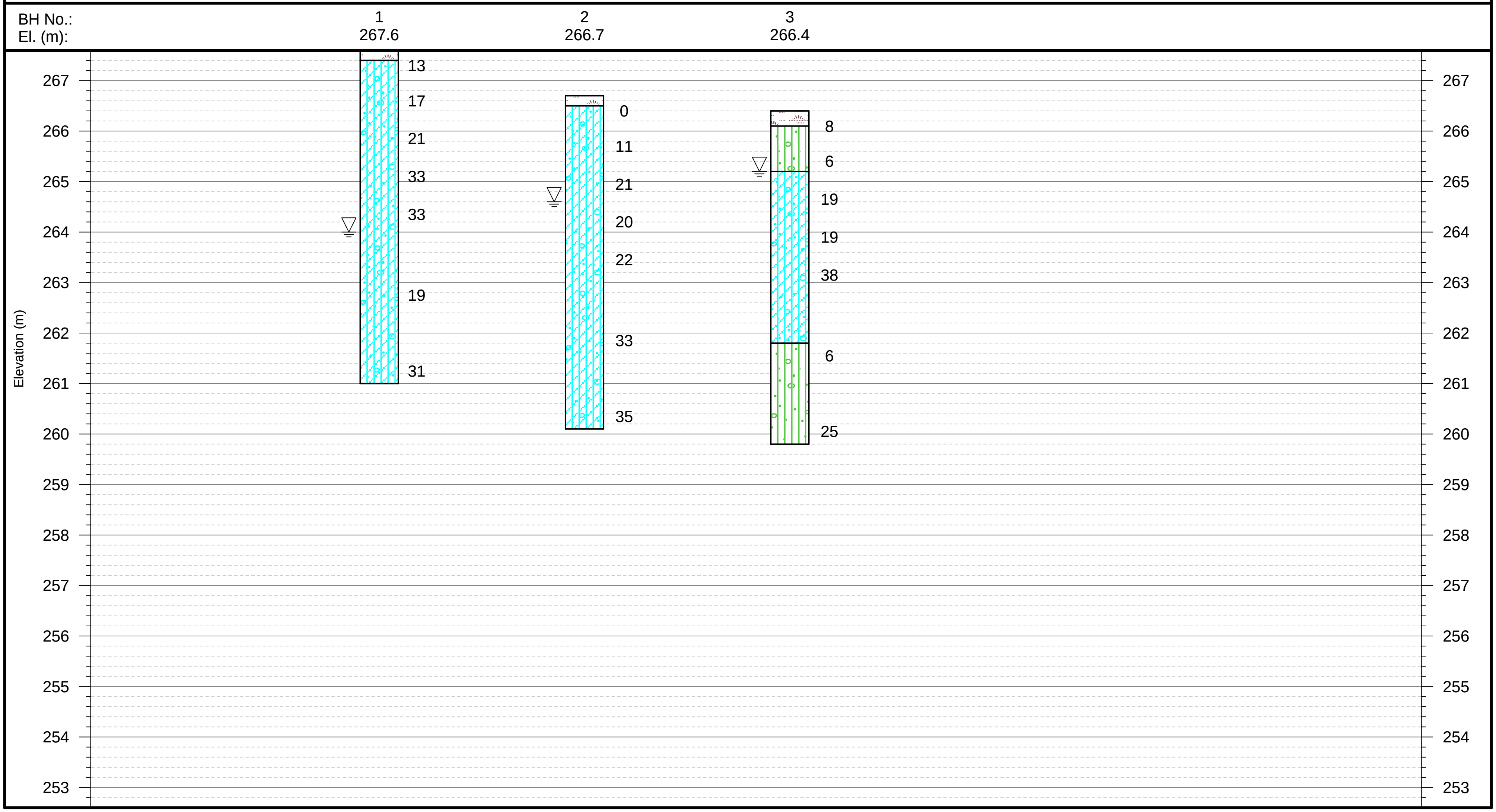
JOB NO.: 2203-S017
REPORT DATE: August 2022
PROJECT DESCRIPTION: Proposed Residential Development

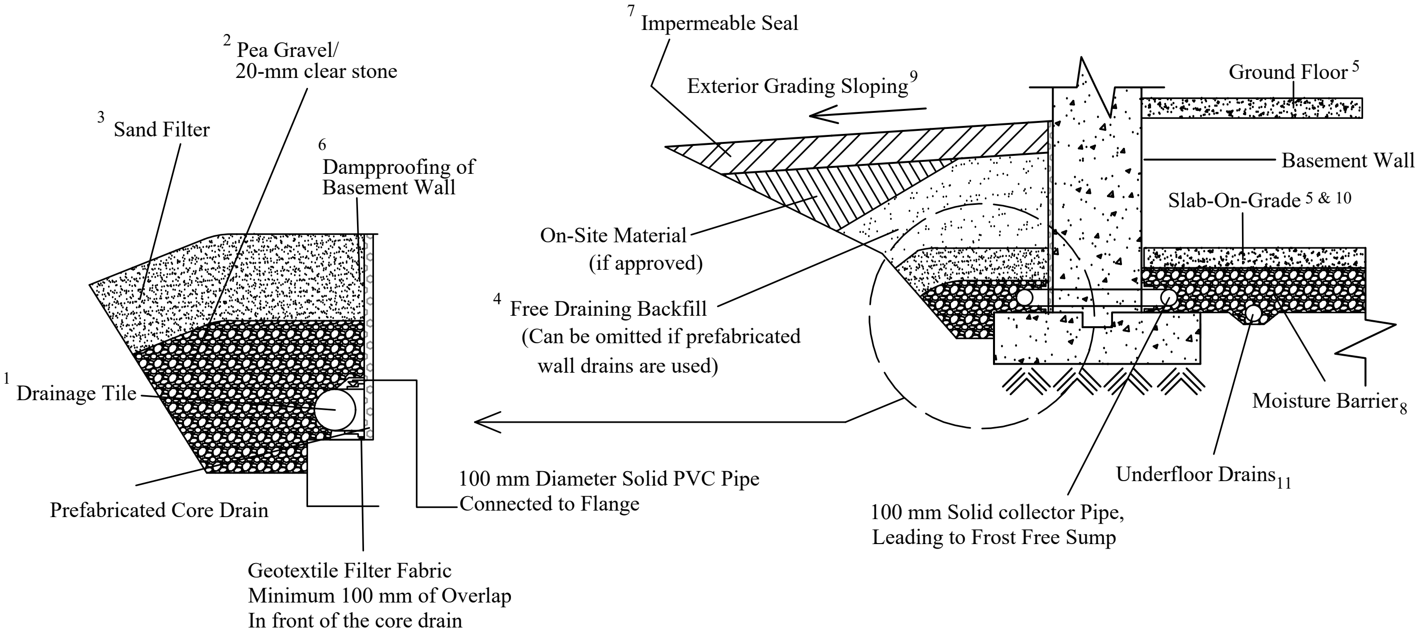
PROJECT LOCATION: 6460 Main Street, Town of Whitchurch-Stoufville

LEGEND

▤ TOPSOIL ▥ SANDY SILT TILL ▨ SILTY CLAY TILL

▽ WATER LEVEL (END OF DRILLING)





NOTES:

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

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**: 19-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 Perimeter Drainage System

SITE 6460 Main Street, Town of Whitchurch-Stoufville

DESIGNED BY T.T. **CHECKED BY** B.L. **DWG NO.** 3

SCALE N.T.S. **REF. NO.** 2203-S017 **DATE** August 2022 **REV** -

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BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	GRAVENHURST	HAMILTON
TEL: (705) 721-7863	TEL: (905) 542-7605	TEL: (905) 440-2040	TEL: (905) 853-0647	TEL: (705) 684-4242	TEL: (905) 777-7956
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

GROUNDWATER MONITORING DATA

REFERENCE NO. 2203-S017

TABLE A-1 MONITOR DETAILS AND GROUNDWATER LEVELS

Project: 6460 Main St. Stouffvile Water Balance (22002.00)

Date	BH22-1			BH22-2			BH22-3		
	WL Depth (mbtoc)	WL Depth (mbgl)	WL Depth (masl)	Water level (mbtoc)	Water level (mbgl)	WL Depth (masl)	Water level Elev (masl)	Water level (mbgl)	WL Depth (masl)
Well Depth			6.10						
17-Mar-22	0.90	-0.10	267.74	1.02	0.00	266.90	2.66	1.67	264.64
22-Mar-22	0.91	-0.09	267.73	1.05	0.04	266.86	2.51	1.52	264.79
3-Apr-22	1.00	0.00	267.64	1.14	0.13	266.77	2.62	1.63	264.68
5-May-22	0.96	-0.04	267.68	1.14	0.13	266.77	2.61	1.62	264.69
15-Jun-22	1.40	0.40	267.24	1.74	0.74	266.90	2.70	1.70	265.94

2022-06-1610:15 AMC:\Users\GARY\Documents\Projects\2022\22002 Stuoffville WB\00 Water Balance\Tech\TAble A-1 GW Levels