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A REPORT TO
BOLTON MIDTOWN DEVELOPMENTS INC.

A GEOTECHNICAL INVESTIGATION AND SLOPE STABILITY ASSESSMENT FOR PROPOSED RESIDENTIAL DEVELOPMENT

13247 AND 13233 NUNNVILLE ROAD

TOWN OF CALEDON (BOLTON)

REFERENCE NO. 1905-S182

JULY 2019

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1.0 INTRODUCTION

In accordance with written authorization dated May 24, 2019 from Mr. Sam Morra of Bolton Midtown Developments Inc., a geotechnical investigation was carried out at 13247 and 13233 Nunnville Road, in the Town of Caledon, for a proposed Residential Development.

The purpose of the investigation was to reveal the subsurface conditions and determine the engineering properties of the disclosed soils for the design and construction of the proposed project.

The geotechnical findings and resulting recommendations are presented in this Report.

2.0 SITE AND PROJECT DESCRIPTION

The Town of Caledon is situated on Peel-Markham till plain where the drift dominates the soil stratigraphy. In places, lacustrine sand, silt, clay and drift which has been reworked by the water action of Peel Ponding (glacial lake) have modified the drift stratigraphy.

The subject site consists of an irregularly shaped parcel of land located at 13247 and 13233 Nunnville Road in the community of Bolton. At the time of the investigation, the properties consisted of a 1- or 2-storey brick dwelling at each property with associated asphalt driveways and septic beds. The remainder of the site is grass-covered with a few scattered trees near the houses as well as at the border of the 2 properties. The ground surface at the site is on a gentle incline, with the higher ground elevation located towards Nunnville Road. The rear of the properties backs onto the adjacent Albion Vaughan Road.

The ground surface along the north boundary of the site descends towards Old King Road at an average gradient of 3.2 to 3.3 horizontal (H):1 vertical (V) at the steepest portions. The slope is approximately 22 to 28 m high, and is densely treed and weed-covered. Houses are located at the bottom of the slope on the south side of Old King Road.

The 2 existing houses at the site are to be demolished for the proposed development, and the site will be subdivided into 35 residential lots. The development will be provided with municipal services and a roadway connecting to Nunnville Road.

3.0 FIELD WORK

The field work, consisting of 5 boreholes to depths of 6.6 to 27.9 m, and 4 groundwater monitoring wells, was performed on June 11 to 14, and 21, 2019, at the locations shown on the Borehole and Cross-Section Location Plan, Drawing No. 1.

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

As mentioned above, 4 groundwater monitoring wells, 50-mm in diameter, were installed at or near 4 of the boreholes to facilitate a slope stability assessment and a hydrogeological study. The findings and assessment for the hydrogeological study will be presented under separate cover.

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

The elevation at each of the borehole and well locations was surveyed by R-PE Surveying Ltd. except for the well near Borehole 2, which was installed on June 21, 2019 after the survey was completed; the elevation of this well was interpolated from the survey plan prepared by R-PE Surveying Ltd.

4.0 SUBSURFACE CONDITIONS

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

The investigation has disclosed that beneath a topsoil layer, and a layer of earth fill in places, the site is underlain by a predominant stratum of silty clay till interstratified with a deposit of silty clay at various locations and depths. Deposits of silt and silty fine sand were encountered in the lower zone of the deep borehole (Borehole 1).

4.1 **Topsoil** (All Boreholes)

The revealed topsoil layer is approximately 10 to 23 cm thick. It is dark brown in colour, indicating that it contains appreciable amounts of roots and humus. These materials are compressible under loads; therefore, the topsoil is considered to be void of engineering value. Due to its humus content, the topsoil will generate an offensive odour and may produce volatile gases under anaerobic conditions. Therefore, the topsoil must not be buried below any structures or deeper than 1.2 m below the exterior finished grade so it will not have an adverse impact on the environmental well-being of the developed area.

Topsoil thicker than that found in the boreholes may occur in places, particularly near low-lying areas, swales and treed areas. In order to prevent overstripping, diligent control of the stripping operation will be required.

Since the topsoil is void of engineering value, it can only be used for general landscape contouring purposes. Its suitability for planting and sodding purposes can be further assessed by fertility testing.

4.2 **Earth Fill** (Boreholes 3 and 4)

The earth fill was encountered beneath the topsoil and extends to depths of 1.9± m and 0.9± m below the prevailing ground surface at Boreholes 3 and 4, respectively. It consists primarily of silty clay, with varying amounts of sand and gravel, and contains organic inclusions.

The obtained 'N' values are 3, 4, 10 and 13 blows per 30 cm of penetration, indicating that the earth fill was loosely placed and the lower portion of the earth fill has self-consolidated.

The natural water content of the samples are 17%, 18% and 30%, indicating that the earth fill is in a moist to wet condition.

Due to the unknown history of the earth fill, and the presence of organic inclusions, the fill is unsuitable for supporting any structures in its current condition. In using the fill for structural backfill, or in pavement or slab-on-grade construction, it should be subexcavated, inspected, sorted free of topsoil inclusions and any deleterious materials, aerated and properly recompacted in thin lifts. If it is impractical to sort the topsoil and other deleterious materials from the fill, the fill must be wasted and replaced with properly compacted inorganic earth fill.

The fill is amorphous in structure; it will ravel and is susceptible to collapse in steep cuts, particularly if the fill is in a wet condition.

One must be aware that the samples retrieved from boreholes 10 cm in diameter may not be truly representative of the geotechnical and environmental quality of the fill, and do not indicate whether the topsoil beneath the earth fill was completely stripped. This should be further assessed by laboratory testing and/or test pits.

4.3 **Silty Clay Till** (All Boreholes) and **Silty Clay** (Boreholes 1, 2, 4 and 5)

The silty clay till dominated the soil stratigraphy at the site; it was encountered beneath the topsoil and/or earth fill and terminated at the maximum investigated depths at all boreholes except Borehole 1. The till consists of a random mixture of soils; the particle sizes range from clay to gravel, with the clay fraction exerting the dominant influence on its properties. It is embedded with sand and silt seams and layers, cobbles and boulders. The structure of the till is heterogeneous, indicating that it is a glacial deposit. The till within the top 0.7± to 1.5± m from the prevailing ground surface has been permeated with fissures and fractured by the weathering process.

Layers of silty clay were encountered interstratified within the clay till, except at Borehole 1 where it was contacted at the bottom of borehole; it contains a trace of sand. The laminated structure shows that the silty clay is a lacustrine deposit.

The obtained 'N' values for the silty clay till range from 3 per 30 cm to 50 per 15 cm, with a median of 20 per 30 cm, showing the consistency of the clay till is soft to hard, being generally very stiff. The obtained 'N' values for the silty clay range from 8 to 44, with a median of 19 per 30 cm, indicating the relative density of the clay is firm to hard, being generally very stiff. The soft soil is restricted to the weathered zone near the surface.

The Atterberg Limits of 1 representative sample each of the silty clay till and silty clay, and the water content of all of the samples were determined. The results are plotted on the Borehole Logs and summarized below:

	Silty Clay Till	**Silty Clay**
Liquid Limit	28%	40%
Plastic Limit	17%	20%
Natural Water Content	12% to 26% (median 18%)	18% to 25% (median 22%)

The above results show that the silty clay till is a cohesive material with low plasticity while the silty clay is a cohesive material with medium plasticity. The natural water content generally lies below its plastic limit or between its plastic and liquid limits, confirming the generally very stiff consistency of the clay till and clay as disclosed by the ‘N’ values.

Grain size analyses were performed on 1 representative sample each of the silty clay till and silty clay; the results are plotted on Figure 6.

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

- High frost susceptibility and low water erodibilty.
- The silty clay till has low soil-adfreezing potential while the silty clay has high soil-adfreezing potential.
- Low permeability, with an estimated coefficient of permeability of 10^{-7} cm/sec, an estimated percolation rate of more than 80 min/cm, and runoff coefficients of:

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

- Cohesive soils, their shear strength is derived from consistency and augmented by internal friction of the silt. Their strength is moisture dependent and, to a lesser degree, dependent on the soil density.
- They will generally be stable in a relatively steep cut. However, prolonged exposure will allow infiltrating precipitation to saturate the weathered zone and the sand and silt seams and layers; this may lead to localized sloughing.
- Very poor to poor pavement-supportive materials, with an estimated California Bearing Ratio (CBR) value of 3% or less.
- Moderately high corrosivity to buried metal, with an estimated electrical resistivity of 2500 to 3500 ohm·cm.

4.4 **Silt** and **Silty Fine Sand** (Borehole 1)

The silt and silty fine sand deposits were contacted within the lower zone of the revealed soil stratigraphy at Borehole 1; the samples contained varying amounts of clay and gravel. The sorted structure indicates that the silt and silty fine sand are glaciolacustrine deposits.

The obtained ‘N’ values for the silt are 50 per 15 cm, 50 per 13 cm and 50 per 10 cm, and the ‘N’ value for the silty fine sand is 50 per 10 cm; this indicates that the relative density of the deposits is very dense.

The natural water content was determined to be 18%, 19% and 22% for the silt samples, and 11% for the silty fine sand, indicating that the silt is in a wet condition and the silty fine sand is in a moist condition. The silt is water bearing and displayed dilatancy when shaken by hand.

A grain size analysis was performed on 1 representative silt sample; the result is plotted on Figure 7.

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

- High frost susceptibility and high soil-adfreezing potential.
- High water erodibility; they are susceptible to migration through small openings under seepage pressure.
- Soils of high capillarity and water retention capacity.
- Relatively pervious to low permeability, depending on the clay content, with an estimated coefficient of permeability of 10^{-4} to 10^{-6} cm/sec, an estimated percolation rate of 12 to 50 min/cm, and runoff coefficients of:

Slope	
0% - 2%	0.07 to 0.15
2% - 6%	0.12 to 0.20
6% +	0.18 to 0.28

- Frictional soils, their shear strength is derived from internal friction; therefore, its shear strength is density dependent. Due to their dilatancy, the strength of the wet silt and sand is susceptible to impact disturbance; i.e., the disturbance will induce a build-up of pore pressure within the soil mantle, resulting in soil dilation and a reduction of shear strength.

4.5 Compaction Characteristics of the Revealed Soils

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

Table 1 - Estimated Water Content for Compaction

Soil Type	Determined Natural Water Content (%)	Water Content (%) for Standard Proctor Compaction	
		100% (optimum)	**Range for 95% or +**
Earth Fill	17, 18 and 30	15	11 to 20
Silty Clay Till	12 to 26 (median 18)	15 to 17	11 to 22
Silty Clay	18 to 25 (median 22)	17 to 19	13 to 24
Silt	18, 19 and 22	13	8 to 17
Silty Fine Sand	11	11	6 to 16

The above values show that the majority of the in situ soils within the expected depths of construction are suitable for a 95% or + Standard Proctor compaction. However, small portions of the silty clay till, silty clay and earth fill may be too wet or on the wet side and may require aeration or mixing with drier soils prior to structural compaction. Aeration of these materials can be achieved by spreading them thinly on the ground in the dry, warm weather. The earth fill should be sorted free of organic inclusions and any deleterious material prior to structural compaction.

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

When compacting the very stiff to hard silty clay till and silty clay on the dry side of the optimum, the compactive energy will frequently bridge over the chunks in the soils and be transmitted laterally into the soil mantle. Therefore, the lifts must be limited to 20 cm or less (before compaction). It is difficult to monitor the lifts of backfill placed in deep trenches; therefore, it is preferable that the compaction of backfill at depths over 1.0 m below the subgrade be carried out on the wet side of the optimum. This would allow a wider latitude of lift thickness.

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

The presence of boulders in the till will prevent transmission of the compactive energy into the underlying material to be compacted. If an appreciable amount of boulders over 15 cm in size is mixed with the material, it must either be sorted or must not be used for structural backfill and/or construction of engineered fill.

5.0 GROUNDWATER CONDITIONS

The boreholes were checked for the presence of groundwater and the occurrence of cave-in upon their completion. The boreholes were dry and open upon their completion; however, groundwater monitoring wells were installed in or near 4 of the geotechnical boreholes, and their water levels were measured on June 27, 2019. The data are plotted on the Borehole Logs and summarized in Table 2.

Table 2 - Groundwater Levels

Borehole No.	Ground Elevation at Borehole/Well* Location (m)	Soil Colour Changes Brown to Grey Depth (m)	Measured Groundwater Level in the Wells on June 27, 2019	
			Depth at well location (m)	Elevation (m)
1 (MW)	243.3/243.3*	7.6	24.6	218.7
2 (MW)	245.0/244.3*	4.6	1.1	243.2
3 (MW)	243.3/243.3*	6.1	1.4	241.9
4	246.1	4.6	N/A	-
5 (MW)	248.2/248.2*	6.1	4.9	243.3

As mentioned above, no groundwater was detected at the time of the field work in the open boreholes and no cave-in levels were recorded; however, groundwater monitoring wells, 50-mm in diameter, were installed at or near 4 of the boreholes, Boreholes 1, 2, 3 and 5. The stabilized water levels recorded in the wells on June 27, 2019 range from depths of 1.1 to 24.6 m below the prevailing ground surface. The shallow groundwater at some of the boreholes may represent a perched groundwater table from infiltrated precipitation; however, a hydrogeological assessment will be provided under separate cover discussing the groundwater conditions at the site. The groundwater level will fluctuate with the seasons.

The soil colour changed from brown to grey at depths ranging from 4.6± to 7.6± m below the prevailing ground surface; the brown colour indicates that the soils have oxidized.

The groundwater yield from the silty clay till and silty clay will be small and limited in quantity, due to the low permeability of the soils, and the yield, if encountered, from any silt or sand deposits will be moderate to appreciable.

Detailed groundwater condition of the site will be discussed in the hydrogeological report under separate cover.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation has disclosed that beneath a topsoil layer, and a layer of earth fill in places, the site is underlain by a predominant stratum of soft to hard, generally very stiff silty clay till, interstratified with firm to hard, generally very stiff silty clay at various locations and depths. Deposits of very dense silt and silty fine sand were encountered in the lower zone of the deep borehole (Borehole 1). The native soil below the earth fill within the top 0.7± to 1.5± m from the prevailing ground surface has been weathered; the soft soil is restricted to the weathered zone.

No groundwater/cave-in levels were recorded in the open boreholes upon completion of the field work; however, groundwater monitoring wells were installed at or near 4 of the boreholes, of which the stabilized groundwater levels were measured at depths of 1.1 to 24.6 m below the prevailing ground surface on June 27, 2019. The shallow groundwater table in places may represent a perched groundwater table from infiltrated precipitation. The groundwater level will fluctuate with the seasons.

Any groundwater yield from the silty clay till and silty clay will be small and limited in quantity, and can generally be controlled by conventional pumping from sumps. The groundwater yield from any silt or sand layers will be moderate to appreciable. The dewatering requirements for the site should be assessed through the hydrogeological study.

It is understood that the property will be developed into a residential subdivision consisting of 35 residential lots with municipal services and a roadway meeting urban standards. The geotechnical findings which warrant special consideration are presented below:

1. The topsoil is unsuitable for engineering applications and must be removed for the development. It should not be buried below any structures or deeper than 1.2 m below the exterior finished grade. Fertility testing can be carried out to assess the suitability of the topsoil as landscaping material.
2. The earth fill is unsuitable for supporting any structures sensitive to settlement in its current condition. In using the fill for structural backfill, or in pavement or slab-on-grade construction, it should be subexcavated, inspected, sorted free of topsoil inclusions and any deleterious materials, aerated and properly recompacted in thin lifts. If it is impractical to sort the deleterious material from the fill, the fill must be wasted and replaced with properly compacted inorganic earth fill.
3. The native soils below the topsoil, earth fill and weathered soil are suitable for normal spread and strip footing construction. The footing 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.
4. Where cut and fill is required for site grading, it is generally more economical to place engineered fill for normal footings, underground services and pavement construction. Weathered soils and any soft material near the ground surface should be subexcavated and upgraded to engineered fill status by aeration, and should be properly recompacted in layers.
5. Special measures must be implemented in the project construction to minimize the risk of damage to the foundations caused by frost action.
6. A Class 'B' bedding, consisting of compacted 20-mm Crusher-Run Limestone, is recommended for the construction of the underground services. The pipe joints should be leak-proof, or wrapped with an appropriate waterproof membrane.
7. Excavation should be carried out in accordance with Ontario Regulation 213/91.

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

6.1 **Slope Stability Analysis**

A slope stability assessment has been carried out to determine the Long-Term Stable Top of Slope (LTSTOS), which is one of the constraints for establishing the development limit of the project.

The slope stability study focuses on the existing slope located at the north limit of the property which descends towards Old King Road. House are located at the bottom of the slope on the south side of Old King Road. In addition, the Humber River is located north of Old King Road, where the river is estimated to be more than 50 m away from the bottom of slope.

Visual inspection of the slope revealed that the ground surface is densely treed and weed-covered with moss cover in places.

The existing slope has an overall height of approximately 22 to 28 m with an average gradient of 3.2 to 3.3H:1V at the steepest portions.

Five cross-sections, Cross-Section A-A, B-B, C-C, D-D and E-E, were selected for analysis of the slope, as representative of the overall slope profile. The locations of the cross-sections are shown on Drawing No. 1. The slope profiles were interpreted from the contours on the provided survey plan. The subsurface profile was interpreted from the logs for Boreholes 1 and 2, where appropriate.

The groundwater level measured on June 27, 2019 in the groundwater monitoring well at Borehole 1, installed for slope stability purposes, was recorded at El. 218.7 m, and has been modelled as a phreatic surface; the water level gradually tapers towards the bottom of slope.

The slope stability at the cross-sections was analysed using the force-moment-equilibrium criteria of the Bishop Method with the soil strength parameters shown in Table 3.

Table 3 - Soil Strength Parameters

Soil Type	**Unit Weight γ (kN/m^3)**	**Cohesion c (kPa)**	**Internal Friction Angle ϕ (degrees)**
Silty Clay Till	22.0	5	30
Silty Clay	20.5	5	26
Silt	21.0	0	30
Silty Fine Sand	20.5	0	31

The results of the analysis are presented on Drawing Nos. 3 to 7, inclusive, and the minimum Factors of Safety (FOS) are summarized in Table 4.

Table 4 - Minimum Factors of Safety (FOS)

Cross-Section	**FOS**	**Drawing No.**
A-A	2.527	3
B-B	2.297	4
C-C	1.890	5

Table 4 - Minimum Factors of Safety (FOS) (cont'd)

Cross-Section	FOS	Drawing No.
D-D	1.675	6
E-E	1.784	7

The FOS for the existing slope at Cross-Sections A-A, B-B, C-C, D-D and E-E meets the Ontario Ministry of Natural Resources (OMNR) guideline requirement for active land use with FOS exceeding 1.5; therefore, the existing physical top of slope can be considered the stable top of slope.

Considering that the Humber River is more than 50 m away from the bottom of slope, a toe erosion allowance is not required for the study.

The LTSTOS has been established on Drawing No. 1. Furthermore, a development setback for man-made and environmental degradation may be required. This is subject to the requirements of the Toronto and Region Conservation Authority (TRCA).

In order to prevent disturbance of the existing slope, the following geotechnical constraints should be stipulated:

1. The prevailing vegetative cover on the slope must be maintained, since its extraction would deprive the slope of the rooting system that is reinforcement against soil erosion by weathering. If, for any reason, the vegetative cover is stripped, it must be reinstated to its original, or better than its original, protective condition. Restoration with selected native plantings including deep rooting systems which would penetrate the original buried topsoil must be carried out after the development to ensure bank stability.

2. Any leafy topsoil cover on the slope face should not be disturbed, since this provides an insulation and screen against frost wedging and rainwash erosion, or the bare slope surface must be adequately sodded.
3. Grading of the land adjacent to the slope must be such that concentrated runoff is not allowed to drain onto the slope face. Landscaping features which may cause runoff to pond at the top of the slope, such as infiltration trenches, as well as saturating the crown of the bank, must not be permitted.
4. Where development is carried out adjacent to the slope, there are other factors to be considered related to possible human environmental abuse. These include soil saturation from frequent watering to maintain landscaping features, stripping of topsoil or vegetation, dumping of loose fill, and material storage close to the top of slope; none of these should be permitted.

The above recommendations are subject to the approval and requirements of the TRCA.

6.2 Foundations

For the proposed development, it is recommended that the normal spread and strip footings be placed below the topsoil, earth fill and weathered soil onto the sound natural soils and/or engineered fill. As a general guide, the recommended soil pressures for use in the design, together with the corresponding suitable founding levels, are presented in Table 5.

Table 5 - Founding Levels

Borehole No.	**Recommended Soil Bearing Pressures (SLS and ULS), and Suitable Founding Level**	
	150 kPa (SLS) 250 kPa (ULS)	
	Depth (m)	**Elevation (m)**
1	1.0 or +	242.3 or -
2	1.0 or +*	244.0 or -
3	2.5 or +	240.8 or -
4	1.8 or +	244.3 or -
5	1.0 or +	247.2 or -

* Due to the decrease in 'N' values with depth, the recommended soil bearing pressure of 150 kPa (SLS) must be reduced to 100 kPa (SLS) from a depth of 3.5 to 7.6 m below the prevailing ground surface. The size of the spread and strip footings should be limited accordingly.

In areas where foundations are to be extended, it may be more cost effective to subexcavate to a size 30% larger than the designed footing width and fill with lean concrete up to the normal footing elevation immediately after the suitable founding soil is exposed.

The existing earth fill and weathered soil can be subexcavated and replaced with engineered fill. Furthermore, where fill is required to raise the grade, or if extended footings and/or cut and fill is required for the site grading, engineered fill suitable for normal footing construction can be considered. Soil bearing pressures of 150 kPa (SLS) and 250 kPa (ULS) are recommended for footings founded on engineered fill. The fill must be certified by the geotechnical consultant that supervised and inspected the fill placement. Details of engineered fill are provided in Section 6.3 of this report.

The recommended bearing pressures (SLS) incorporate a safety factor of 3. The total and differential settlements of the footings are estimated to be 25 mm and 15 mm, respectively.

The foundation subgrade must be inspected by a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to assess its suitability for bearing the designed foundations.

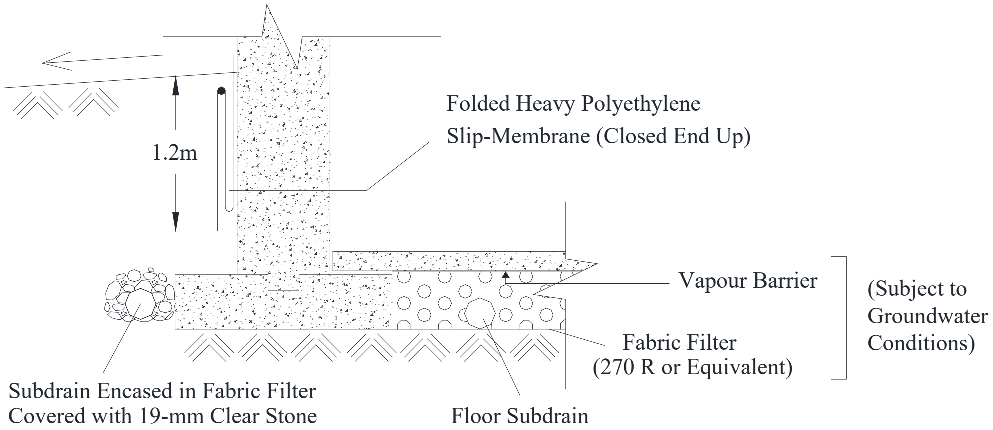
Footings exposed to weathering, and in unheated areas, should have at least 1.2 m of earth cover for protection against frost action.

Perimeter subdrains and dampproofing of the foundation walls will be required. All the subdrains should be encased in a fabric filter to protect them against blockage by silting.

It should be noted that if groundwater seepage is encountered during the footing excavations, or where the subgrade of the normal foundations is found to be wet, the footings must be poured immediately after subgrade inspection or the subgrade should be protected by a concrete mud-slab immediately after exposure. This will prevent construction disturbance and costly rectification of the bearing subsoil.

The foundation walls must be constructed of concrete and either backfilled with non-frost-susceptible pit-run granular, or should be properly shielded with a polyethylene slip-membrane extending below the frost depth to alleviate the risk of frost damage. If the proposed structures have a basement and groundwater seepage is detected at the time of foundation excavation, under-floor subdrains may be installed and they must be connected to sump-wells or to drains which have a positive outlet. Also, a vapour barrier should be installed to prevent upfiltration of soil moisture that may wet the floor. The recommended measures are schematically presented in Diagram 1.

Diagram 1 - Frost Protection Measures (Foundation)



The necessity to implement the above measures should be assessed at the time of construction.

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

6.3 <u>Engineered Fill</u>

The existing earth fill and weathered soil can be upgraded to or replaced with engineered fill, and where earth fill is required to raise the site or extended footings are required, it is generally more economical to place engineered fill for normal footing, underground services and pavement construction. The engineering requirements for a certifiable fill for pavement construction, municipal services, slab-on-grade, and footings designed with soil bearing pressures of 150 kPa (SLS) and 250 kPa (ULS) are presented below:

1. All the existing topsoil must be removed, and the subgrade must be inspected and proof-rolled prior to any fill placement. The existing earth fill and badly weathered soil must be subexcavated, sorted free of topsoil inclusions and deleterious materials, if any, aerated and properly compacted.
2. Inorganic soils must be used for the fill, and they must be uniformly compacted in lifts, 20 cm thick, to 98% or + of their maximum Standard Proctor dry density up to the proposed finished grade and/or slab-on-grade subgrade. The soil moisture must be properly controlled on the wet side of the optimum. If the foundations are to be built soon after the fill placement, the densification process for the engineered fill must be increased to 100% of the maximum Standard Proctor compaction.
3. If imported fill is to be used, it should be inorganic soils, free of any deleterious 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.
4. If the engineered fill is to be left over the winter months, adequate earth cover, or equivalent, must be provided for protection against frost action.
5. The engineered fill must extend over the entire graded area; the engineered fill envelope and finished elevations must be clearly and accurately defined in the field, and they must be precisely documented by qualified surveyors.

   Foundations partially on engineered fill must be reinforced by two 15-mm steel reinforcing bars in the footings and upper section of the foundation walls, or be designed by a structural engineer, to properly distribute the stress induced by the abrupt differential settlement (estimated to be 15± mm) between the natural soils and engineered fill.

6. The engineered fill must not be placed during the period from late November to early April, when freezing ambient temperatures occur either persistently or intermittently. This is to ensure that the fill is free of frozen soils, ice and snow.
7. Where fill is to be placed on a bank steeper than 3H:1V, the face of the bank must flattened to 3+H:1V so that it is suitable for safe operation of the compactor and the required compaction can be obtained.
8. Where the ground is wet due to subsurface water seepage, an appropriate subdrain scheme must be implemented prior to the fill placement, particularly if it is to be carried out on sloping ground.
9. The fill operation must be inspected on a full-time basis by a technician under the direction of a geotechnical engineer.
10. The footing and underground services subgrade must be inspected by the geotechnical consulting firm that inspected the engineered fill placement. This is to ensure that the foundations are placed within the engineered fill envelope, and the integrity of the fill has not been compromised by interim construction, environmental degradation and/or disturbance by the footing excavation.
11. Any excavation carried out in certified engineered fill must be reported to the geotechnical consultant who inspected the fill placement in order to document the locations of the excavation and/or to inspect 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.
12. Despite stringent control in the placement of the engineered fill, variations in soil type and density may occur in the engineered fill. Therefore, the strip footings and the upper section of the foundation walls constructed on the engineered fill may require continuous reinforcement with steel bars, depending on the

uniformity of the soils in the engineered fill and the thickness of the engineered fill underlying the foundations. Should the footings and/or walls require reinforcement, the required number and size of reinforcing bars must be assessed by considering the uniformity as well as the thickness of the engineered fill beneath the foundations. In sewer construction, the engineered fill is considered to have the same structural proficiency as a natural inorganic soil.

6.4 <u>Underground Services</u>

The subgrade for the underground services should consist of sound natural soils or properly compacted organic-free earth fill. Where topsoil, organic earth fill or badly weathered soil is encountered, it should be subexcavated and replaced with properly compacted inorganic soil and/or bedding material compacted to at least 98% or + of its Standard Proctor compaction.

A Class 'B' bedding is recommended for the underground services construction. The bedding material should consist of compacted 20-mm Crusher-Run Limestone, or equivalent. The pipe joints should be leak-proof, or the joints should be wrapped with a waterproof membrane, to prevent subgrade upfiltration through the joints.

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

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

Since the silty clay till and silty clay have moderately high corrosivity to buried metal, any metal fittings and pipes should be protected against soil corrosion. In determining the mode of protection, an electrical resistivity of 2500 ohm·cm should be used. This, however, should be confirmed by testing the soil along the water main alignment at the time of construction.

6.5 Backfilling in Trenches and Excavated Areas

The on site inorganic soils are generally suitable for use as trench backfill. However, the soils should be sorted free of any topsoil inclusions and other deleterious materials prior to the backfilling.

The backfill in trenches and excavated areas should be compacted to at least 95% of its maximum Standard Proctor dry density and increased to 98% or + below the floor slab. In the zone within 1.0 m below the pavement subgrade, the materials should be compacted with the water content 2% to 3% drier than the optimum, and the compaction should be increased to at least 98% of the respective maximum Standard Proctor dry density. This is to provide the required stiffness for pavement construction. In the lower zone, the compaction should be carried out on the wet side of the optimum; this allows a wider latitude of lift thickness.

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

The narrow trenches for services crossings should be cut at 2 or +H:1V so that the backfill can be effectively compacted. Otherwise, soil arching will prevent the achievement of proper compaction. The lift of each backfill layer should either be limited to a thickness of 20 cm, or the thickness should be determined by test strips.

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

- When construction is carried out in freezing winter weather, allowance should be made for these following conditions. Despite stringent backfill monitoring, frozen soil layers may inadvertently be mixed with the structural trench backfill. Should the in situ soils have a water content on the dry side of the optimum, it would be impossible to wet the soils due to the freezing condition, rendering difficulties in obtaining uniform and proper compaction. Furthermore, the freezing condition will prevent flooding of the backfill when it is required, such as in a narrow vertical trench section, or when the trench box is removed. The above will invariably cause backfill settlement that may become evident within 1 to several years, depending on the depth of the trench which has been backfilled.
- In areas where the construction is carried out during the winter months, prolonged exposure of the trench walls will result in frost heave within the soil mantle of the walls. This may result in some settlement as the frost recedes, and repair costs will be incurred prior to final surfacing of the new pavement and the slab-on-grade construction.
- In deep trench backfill, one must be aware that future settlement is to be expected, unless the side of the cut is flattened to at least 1.5+H:1V, and the lifts of the fill and its moisture content are stringently controlled; i.e., lifts should be no more than 20 cm (or less if the backfilling conditions dictate) and uniformly

compacted to achieve at least 95% of the maximum Standard Proctor dry density, with the moisture content on the wet side of the optimum.

- It is often difficult to achieve uniform compaction of the backfill in the lower vertical section of a trench which is an open cut or is stabilized by a trench box, particularly in the sector close to the trench walls or the sides of the box. These sectors must be backfilled with sand. In a trench stabilized by a trench box, the void left after the removal of the box will be filled by the backfill. It is necessary to backfill this sector with sand, and the compacted backfill must be flooded for 1 day, prior to the placement of the backfill above this sector; i.e., in the upper sloped trench section. This measure is necessary in order to prevent consolidation of inadvertent voids and loose backfill which will compromise the compaction of the backfill in the upper section. In areas where groundwater movement is expected in the sand fill mantle, anti-seepage collars should be provided.

6.6 **Garages, Driveways, Sidewalks, Interlocking Stone Pavement and Landscaping**

Due to the high frost susceptibility of some of the underlying soils, heaving of the pavement is expected to occur during the cold weather.

The driveways leading to the garages should be backfilled with non-frost susceptible granular material, with a frost taper at a slope of 1H:1V.

The garage floor slab and interior garage foundation walls should be insulated with 50-mm Styrofoam, or equivalent.

It is recommended that interlocking stone pavement, sidewalks and landscaping structures in areas which are sensitive to frost-induced ground movement be constructed on a free-draining, non-frost-susceptible granular material, and be provided with positive drainage, such as weeper subdrains connected to manholes or catch basins.

6.7 Pavement Design

The recommended pavement design for local roads is presented in Table 6.

Table 6 - Pavement Design

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	65	HL-8
Granular Base	150	Granular ‘A’ or equivalent
Granular Sub-base	300	Granular ‘B’, or equivalent

In preparation of the subgrade, the topsoil should be removed and the subgrade surface must be proof-rolled. The existing earth fill, weathered soil or soft 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 98% or + of its maximum Standard Proctor dry density.

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

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

The road subgrade will suffer a strength regression if water is allowed to saturate the mantle. The following measures should, therefore, be incorporated into the construction procedures and pavement design:

- If the road construction does not immediately follow the trench backfilling, the subgrade should be properly crowned and smooth-rolled to allow interim precipitation to be properly drained.
- Areas adjacent to the road should be properly graded to prevent ponding of large amounts of water during the interim construction period.
- Curb subdrains will be required. The subdrains should consist of filter-sleeved weepers to prevent blockage by silting.
- If the road is to be constructed during wet seasons and extensively soft subgrade occurs, the granular sub-base should be thickened in order to compensate for the inadequate strength of the subgrade. This can be assessed during construction.

6.8 **Stormwater Infiltration Potential**

Based on the borehole findings, the site is primarily underlain by a stratum of silty clay till with silty clay. The estimated permeability of the clayey soils is 10^{-7} cm/sec, with an estimated percolation rate of over 80 min/cm. In general, infiltration of the rainwater is not practical where the subsoil consists of impervious clay till or clay.

Any percolated water in the ground tends to move horizontally, being intercepted by subdrains, swales or ditches, which will eventually be drained into the storm sewer.

Due to the low permeability nature of the encountered soils on site, the potential for infiltration practice is low for this site.

The estimated percolation rates are based on gradation analysis and are provided as a guideline only.

Infiltration galleries, if any, must not be located at or near the top of slope to prevent impacting the stability of the slope.

6.9 Soil Parameters

The recommended soil parameters for the project design are given in Table 7.

Table 7 - Soil Parameters

Unit Weight and Bulk Factor	**Unit Weight (kN/m^3)**		**Estimated Bulk Factor**	
	Bulk	**Submerged**	**Loose**	**Compacted**
Earth Fill	20.5	10.5	1.20	0.98
Silty Clay Till	22.0	12.5	1.33	1.03
Silty Clay	20.5	11.5	1.30	1.00
Silt	21.0	10.5	1.20	1.00
Silty Fine Sand	20.5	10.8	1.20	0.98

Table 7 - Soil Parameters (cont’d)

Lateral Earth Pressure Coefficients	**Active K_a**	**At Rest K_0**	**Passive K_p**
Earth Fill and Silty Clay	0.40	0.55	2.50
Silty Clay Till, Silt and Silty Fine Sand	0.33	0.48	3.00

Coefficients of Friction	
Between Concrete and Granular Base	0.60
Between Concrete and Sound Natural Soils	0.40

6.10 Excavation

Excavation should be carried out in accordance with Ontario Regulation 213/91.

Excavations in excess of 1.2 m should be sloped at 1H:1V for stability. In earth fill, weathered soil and/or where groundwater is encountered, the sides of excavations may need to be flattened to 1.5 or +H:1V for stability.

For excavation purposes, the types of soils are classified in Table 8.

Table 8 - Classification of Soils for Excavation

Material	Type
Sound natural Clay Till and Clay	2
Weathered Soil, and dewatered Silt and Sand	3
Saturated Silt and Sand	4

Excavation into the till containing boulders may require extra effort and the use of a heavy-duty backhoe. Boulders larger than 15 cm in size are not suitable for structural backfill and/or construction of engineered fill.

The groundwater yield from the silty clay till and silty clay will be small and limited in quantity, due to the low permeability of the soils, and can generally be controlled by conventional pumping from sumps. The yield, if encountered, from any silt or sand deposits will be moderate to appreciable. The dewatering requirements for the site should be assessed through the hydrogeological study.

Prospective contractors must be asked to assess the in situ subsurface conditions for soil cuts by digging test pits to at least 0.5 m below the intended bottom of excavation. These test pits should be allowed to remain open for a period of at least 4 hours to assess the trenching conditions.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the accounts of Bolton Midtown Developments Inc., and for review by its designated consultants and government agencies. Use of the report is subject to the conditions and limitations of the contractual agreement.

The material in the report reflects the judgement of Mumta Mistry, B.A.Sc., and 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, are the responsibility of such Third Parties. Soil Engineers Ltd. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

SOIL ENGINEERS LTD.

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LICENSED PROFESSIONAL ENGINEER
July 26/2019
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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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CONSULTING ENGINEERS
GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE

JOB NO.: 1905-S182

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

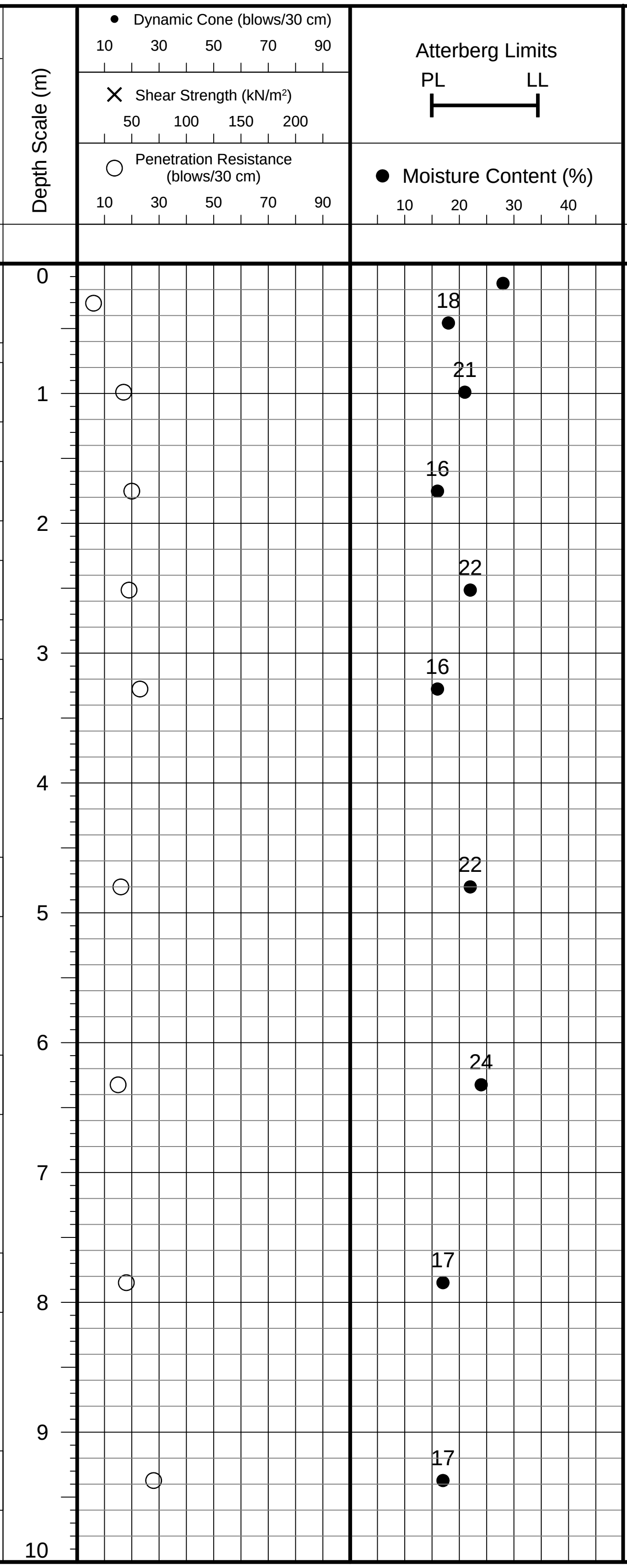
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Tricone

PROJECT LOCATION: 13247 and 13233 Nunnville Road
Town of Caledon (Bolton)

DRILLING DATE: June 11 to 13, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
243.3	Ground Surface			
0.0	**15 cm TOPSOIL**	1A	DO	6
	Firm to very stiff **SILTY CLAY TILL** a trace to some sand a trace of gravel occ. wet sand and silt seams and layers, cobbles and boulders organic inclusions weathered	1B		
		2	DO	17
		3	DO	20
		4	DO	19
		5	DO	23
		6	DO	16
		7	DO	15
	brown grey	8	DO	18
		9	DO	28
233.3	*(Continued on next page)*			



WATER LEVEL

No W.L. on completion of field work due to use of water to drill borehole

Soil Engineers Ltd.

JOB NO.: 1905-S182

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

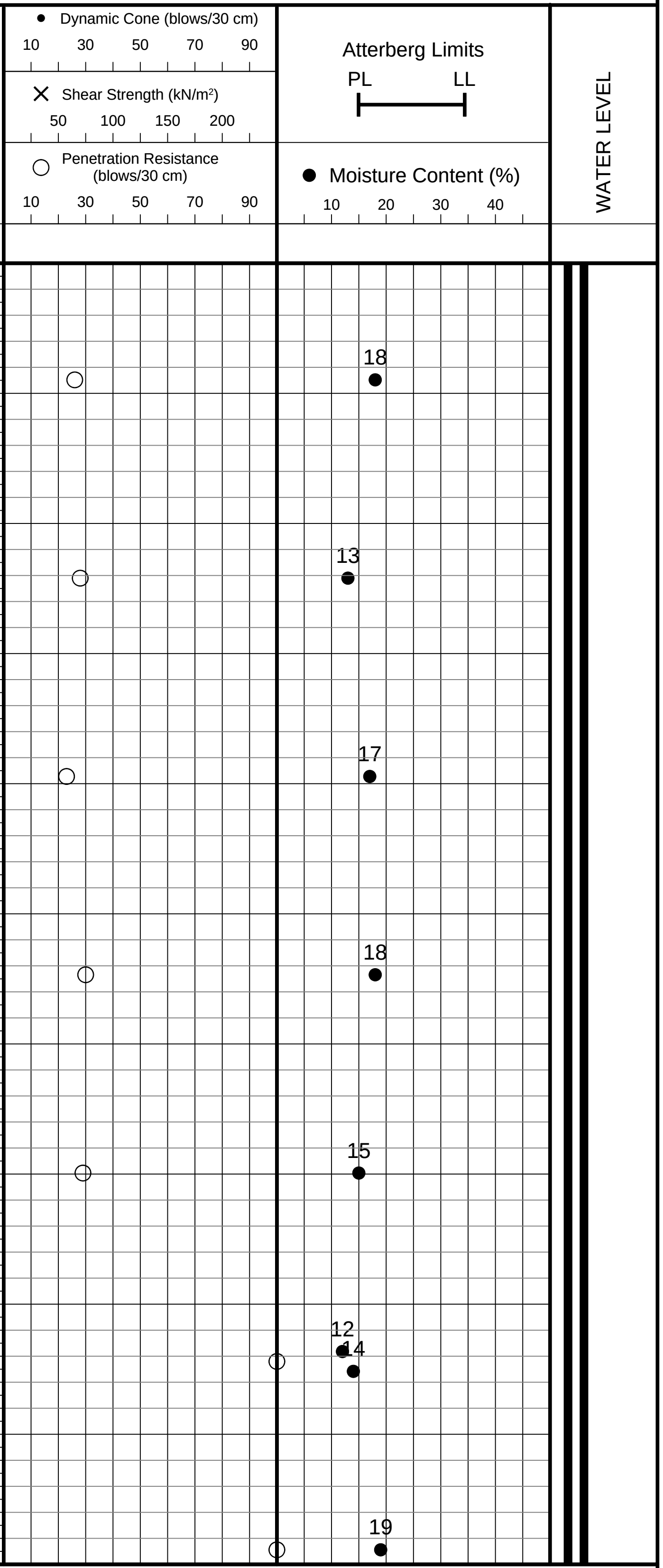
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Tricone

PROJECT LOCATION: 13247 and 13233 Nunnville Road
Town of Caledon (Bolton)

DRILLING DATE: June 11 to 13, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)
10.0	*(Continued)* Grey, very stiff to hard **SILTY CLAY TILL** a trace to some sand a trace of gravel occ. wet sand and silt seams and layers, cobbles and boulders				10
		10	DO	26	11
		11	DO	28	12
		12	DO	23	13 14
		13	DO	30	15
		14	DO	29	16 17
	— grey brown	15A 15B	DO	50/15	18
223.5					19
19.8 223.3	Grey **SILT** *(Continued on next page)*	16	DO	50/15	20



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JOB NO.: 1905-S182

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed Residential Development

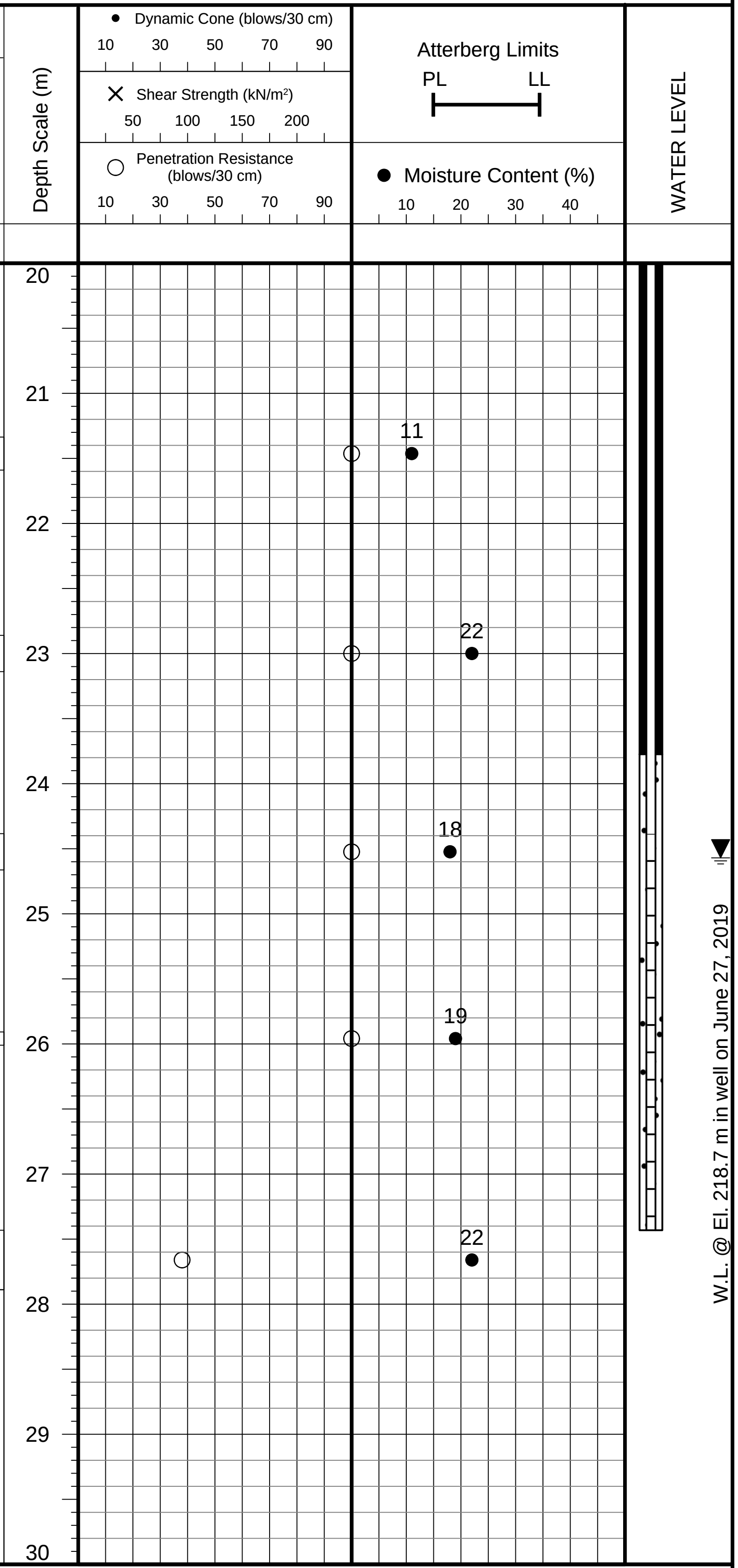
METHOD OF BORING: Tricone

PROJECT LOCATION: 13247 and 13233 Nunnville Road
Town of Caledon (Bolton)

DRILLING DATE: June 11 to 13, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
20.0	*(Continued)* Grey, very dense **SILT** some clay and sand a trace of gravel			
222.0 21.3	Brown, very dense **SILTY FINE SAND** a trace of clay	17	DO	50/10
220.4 22.9	Brown, very dense **SILT** traces to some clay and sand a trace of gravel	18	DO	50/13
		19	DO	50/13
		20	DO	50/10
215.9 27.4	Grey, hard **SILTY CLAY** a trace of sand	21	DO	38
215.4 27.9	**END OF BOREHOLE**			

Installed 50 mm Ø PVC monitoring well to 27.4 m (3.0 m screen)
Sand backfill from 23.8 to 27.4 m
Bentonite holeplug from 0.0 to 23.8 m
Provided with a 4x4 steel monument casing with top and bottom caps, and a lock



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JOB NO.: 1905-S182

LOG OF BOREHOLE NO.: 2

FIGURE NO.: 2

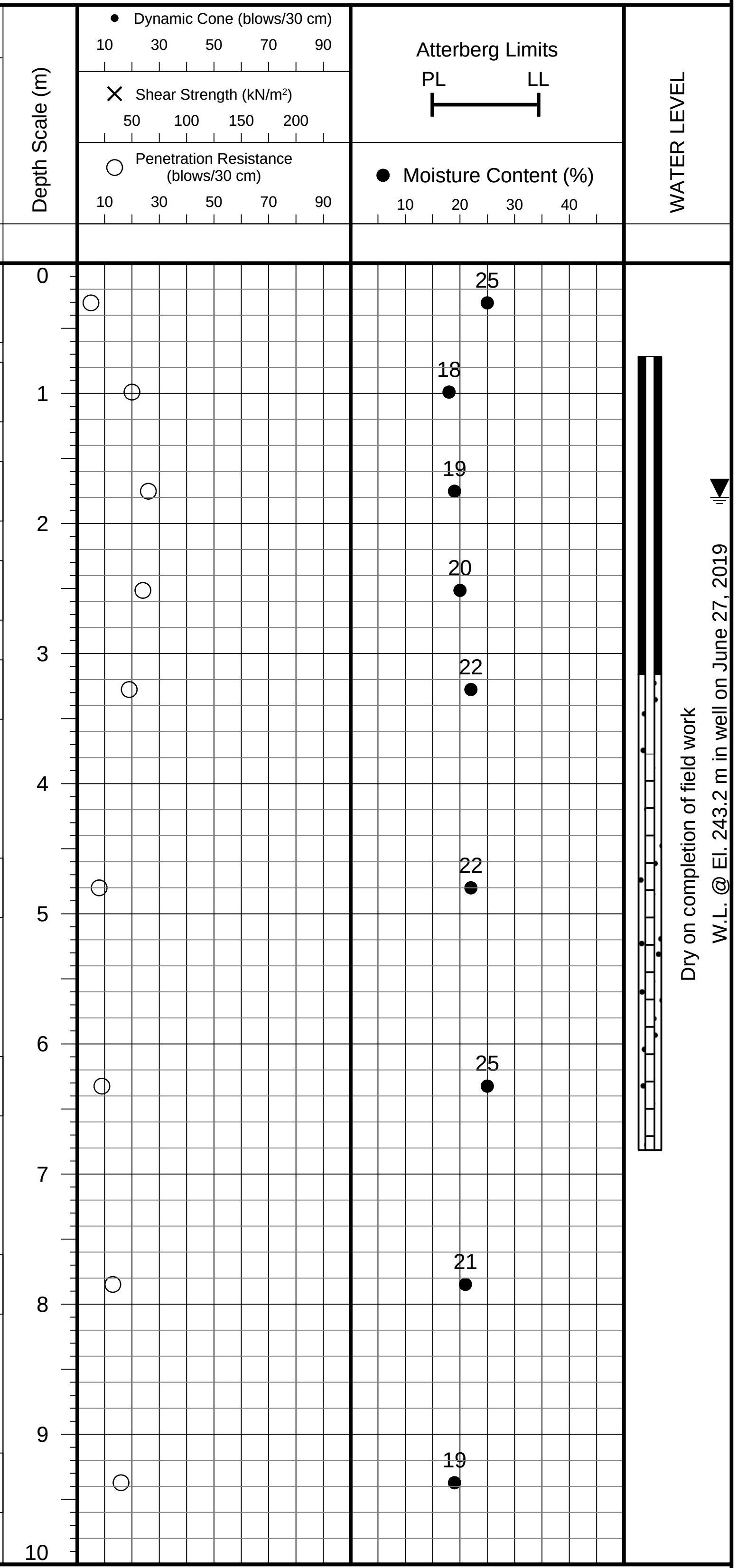
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Solid Stem Auger

PROJECT LOCATION: 13247 and 13233 Nunnville Road
Town of Caledon (Bolton)

DRILLING DATE: June 13 and 21, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
245.0	Ground Surface			
0.0	**10 cm TOPSOIL** Brown, firm to very stiff **SILTY CLAY TILL** organic inclusions/ weathered a trace to some sand a trace of gravel occ. wet sand and silt seams and layers, cobbles and boulders	1	DO	5
		2	DO	20
		3	DO	26
		4	DO	24
		5	DO	19
240.4 4.6	Grey, firm to stiff **SILTY CLAY** a trace of sand	6	DO	8
		7	DO	9
237.4 7.6	Grey, stiff **SILTY CLAY TILL** a trace to some sand a trace of gravel occ. wet sand and silt seams and layers, cobbles and boulders	8	DO	13
		9	DO	16
235.0	*(Continued on next page)*			



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JOB NO.: 1905-S182

LOG OF BOREHOLE NO.: 2

FIGURE NO.: 2

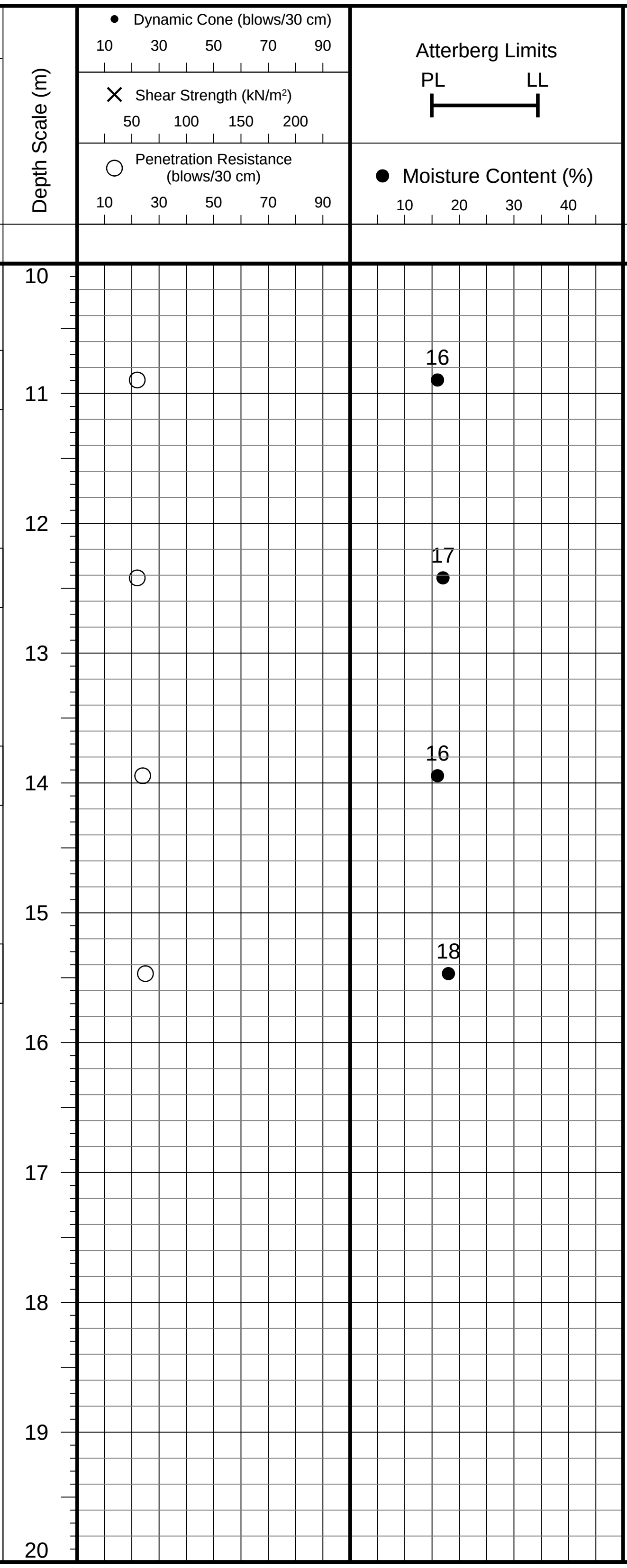
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Solid Stem Auger

PROJECT LOCATION: 13247 and 13233 Nunnville Road
Town of Caledon (Bolton)

DRILLING DATE: June 13 and 21, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value	WATER LEVEL
10.0	*(Continued)* Grey, very stiff **SILTY CLAY TILL** a trace to some sand a trace of gravel occ. wet sand and silt seams and layers, cobbles and boulders	10	DO	22	
		11	DO	22	
		12	DO	24	
229.3 15.7		13	DO	25	
	END OF BOREHOLE Well installed approximately 10 m northeast of the borehole Ground surface elevation at well location = 244.3 m Installed 50 mm Ø PVC monitoring well to 6.1 m (3.0 m screen) Sand backfill from 2.4 to 6.1 m Bentonite holeplug from 0.0 to 2.4 m Provided with a 4x4 steel monument casing with top and bottom caps, and a lock				



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JOB NO.: 1905-S182

LOG OF BOREHOLE NO.: 3

FIGURE NO.: 3

PROJECT DESCRIPTION: Proposed Residential Development

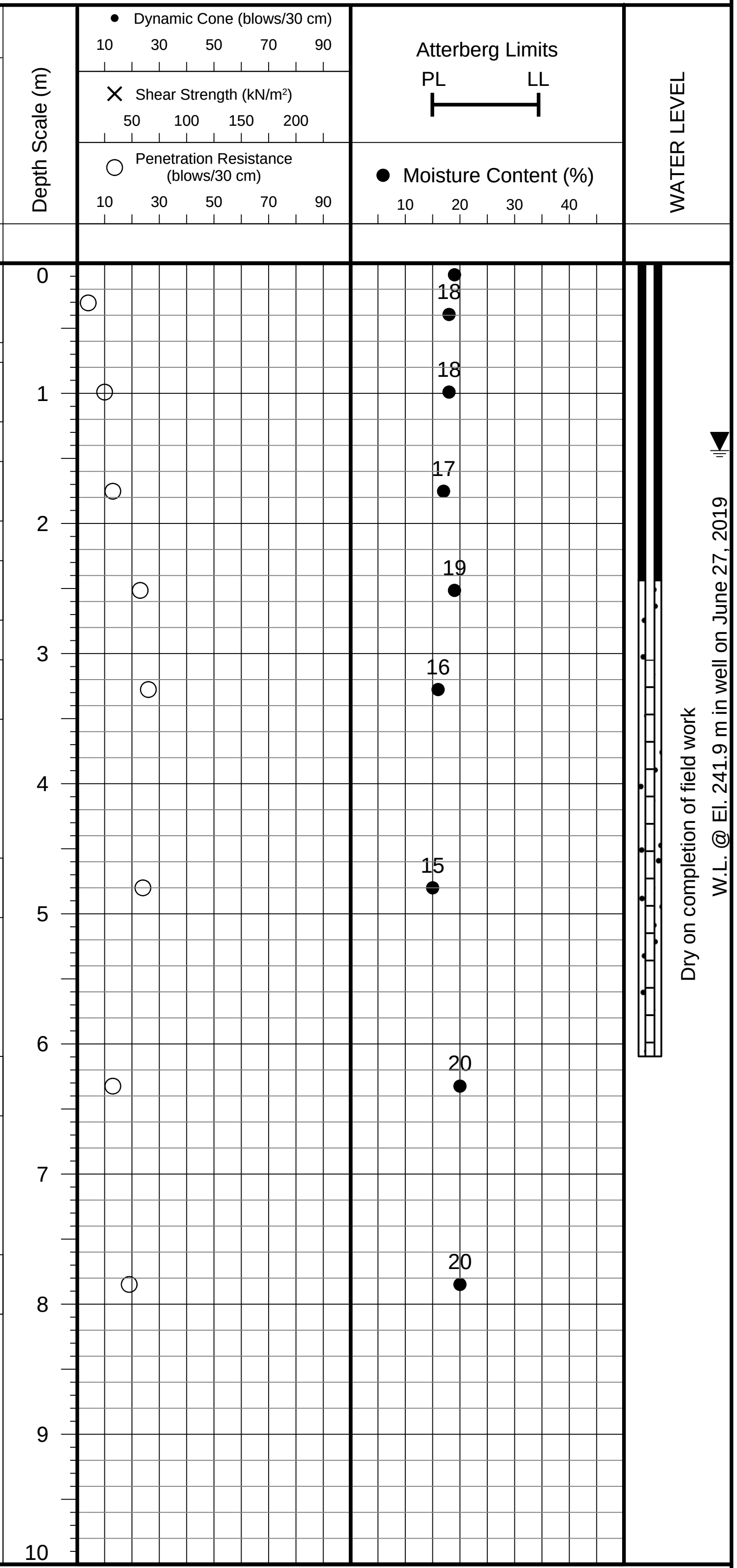
METHOD OF BORING: Solid Stem Auger

PROJECT LOCATION: 13247 and 13233 Nunnville Road
Town of Caledon (Bolton)

DRILLING DATE: June 14, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
243.3	Ground Surface			
0.0	**18 cm TOPSOIL**	1A	DO	4
	Brown/grey **EARTH FILL (Silty Clay)** traces of sand and gravel	1B		
		2	DO	10
		3	DO	13
241.4 1.9	Stiff to very stiff **SILTY CLAY TILL** a trace to some sand a trace of gravel occ. wet sand and silt seams and layers, cobbles and boulders			
		4	DO	23
		5	DO	26
		6	DO	24
	brown grey	7	DO	13
		8	DO	19
235.2 8.1	**END OF BOREHOLE**			

Installed 50 mm Ø PVC monitoring well to 6.1 m (3.0 m screen)
Sand backfill from 2.4 to 6.1 m
Bentonite holeplug from 0.0 to 2.4 m
Provided with a 4x4 steel monument casing with top and bottom caps, and a lock



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JOB NO.: 1905-S182

LOG OF BOREHOLE NO.: 4

FIGURE NO.: 4

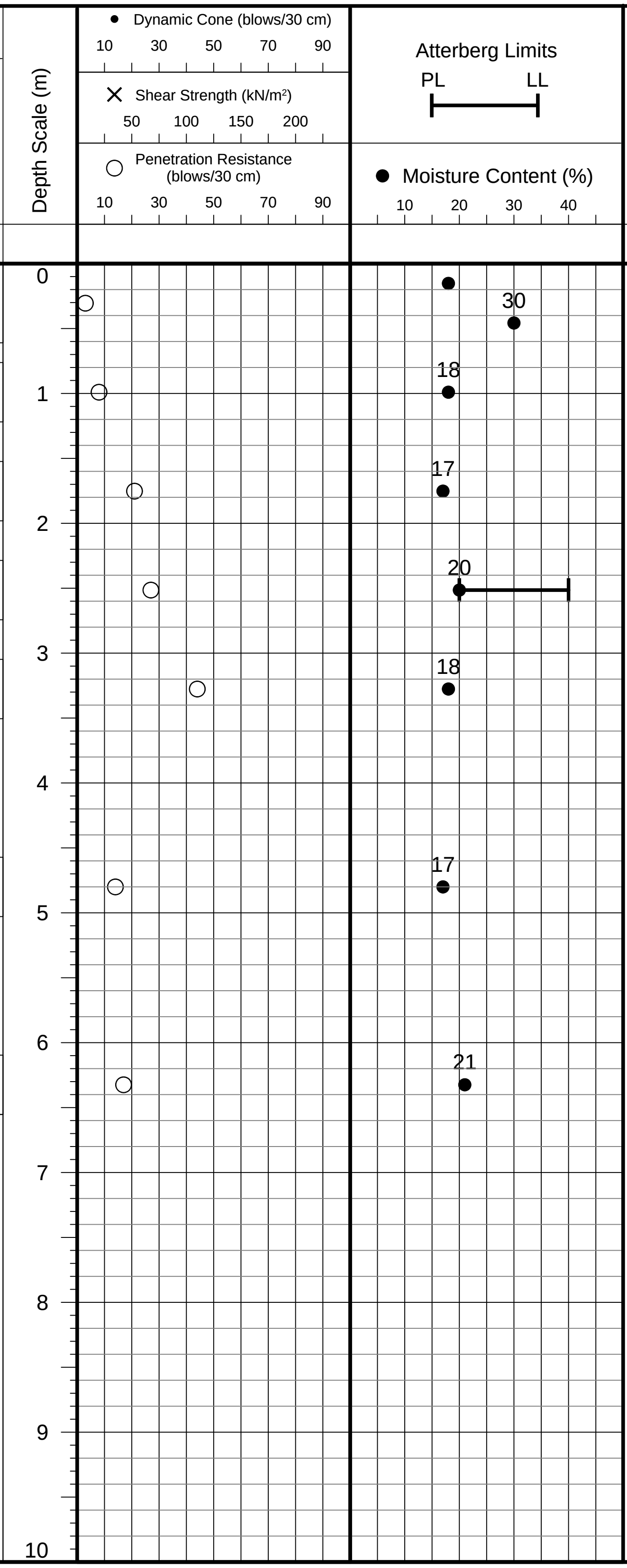
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Solid Stem Auger

PROJECT LOCATION: 13247 and 13233 Nunnville Road
Town of Caledon (Bolton)

DRILLING DATE: June 14, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value	WATER LEVEL
246.1	Ground Surface				
0.0	**20 cm TOPSOIL**	1A	DO	3	Dry on completion of field work
	Brown **EARTH FILL** **(Silty Clay)** some sand, a trace of gravel with organic inclusions	1B			
245.2 0.9	Brown, firm to very stiff **SILTY CLAY TILL** traces of sand and gravel occ. wet sand and silt seams and layers, cobbles and boulders	2	DO	8	
	weathered	3	DO	21	
243.8 2.3	Brown, very stiff to hard **SILTY CLAY** a trace of sand	4	DO	27	
		5	DO	44	
241.5 4.6	Grey, stiff to very stiff **SILTY CLAY TILL** traces of sand and gravel occ. wet sand and silt seams and layers, cobbles and boulders	6	DO	14	
		7	DO	17	
239.5 6.6	**END OF BOREHOLE**				



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JOB NO.: 1905-S182

LOG OF BOREHOLE NO.: 5

FIGURE NO.: 5

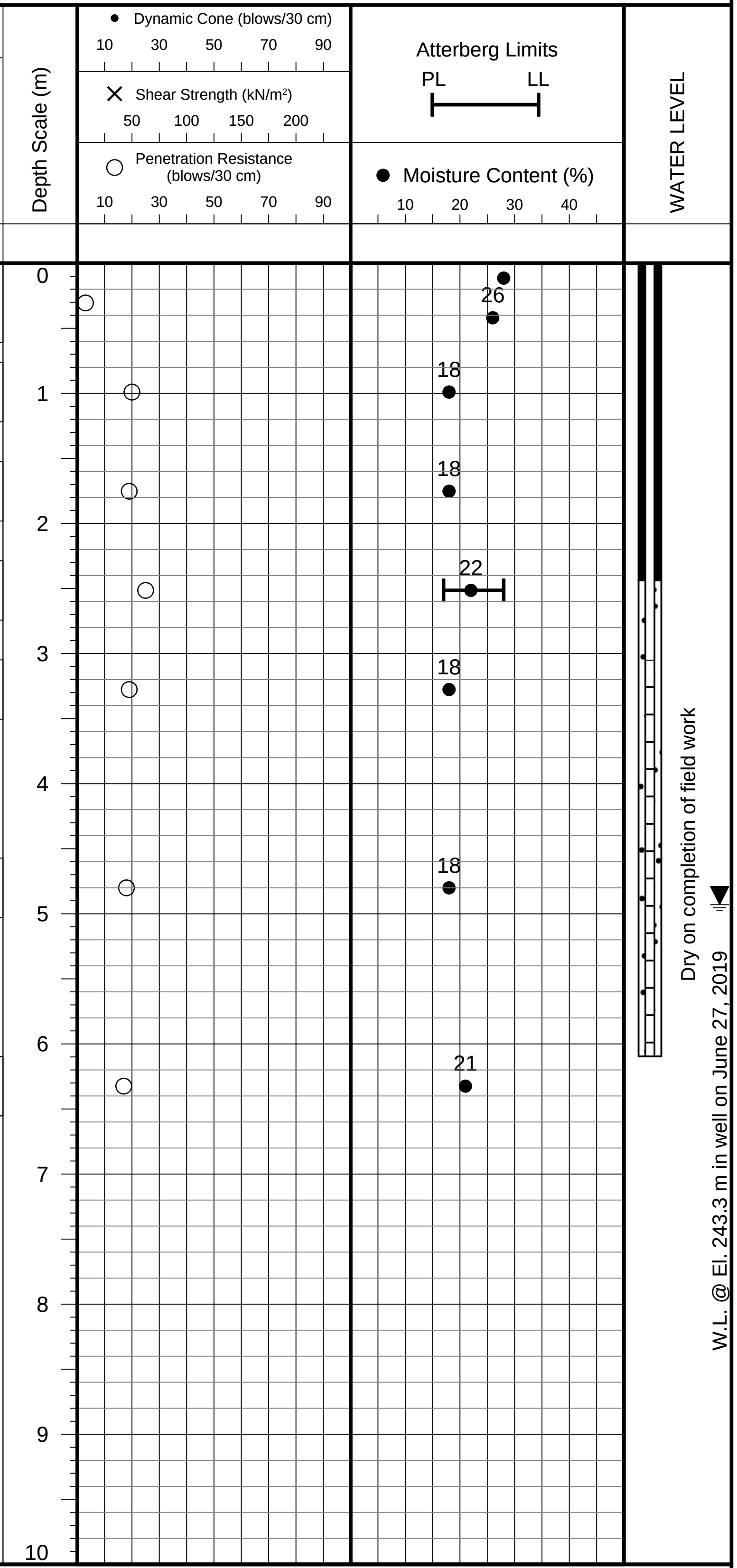
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Solid Stem Auger

PROJECT LOCATION: 13247 and 13233 Nunnville Road
Town of Caledon (Bolton)

DRILLING DATE: June 14, 2019

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
248.2	Ground Surface			
0.0	**23 cm TOPSOIL**	1A	DO	3
	Brown, soft to very stiff **SILTY CLAY TILL** a trace to some sand a trace of gravel occ. wet sand and silt seams and layers, cobbles and boulders organic inclusions/ weathered	1B		
		2	DO	20
246.7 1.5	Brown, very stiff **SILTY CLAY** a trace of sand	3	DO	19
245.9 2.3	Very stiff **SILTY CLAY TILL** a trace to some sand a trace of gravel occ. wet sand and silt seams and layers, cobbles and boulders	4	DO	25
		5	DO	19
		6	DO	18
	brown grey	7	DO	17
241.6 6.6	**END OF BOREHOLE** Installed 50 mm Ø PVC monitoring well to 6.1 m (3.0 m screen) Sand backfill from 2.4 to 6.1 m Bentonite holeplug from 0.0 to 2.4 m Provided with a 4x4 steel monument casing with top and bottom caps, and a lock			



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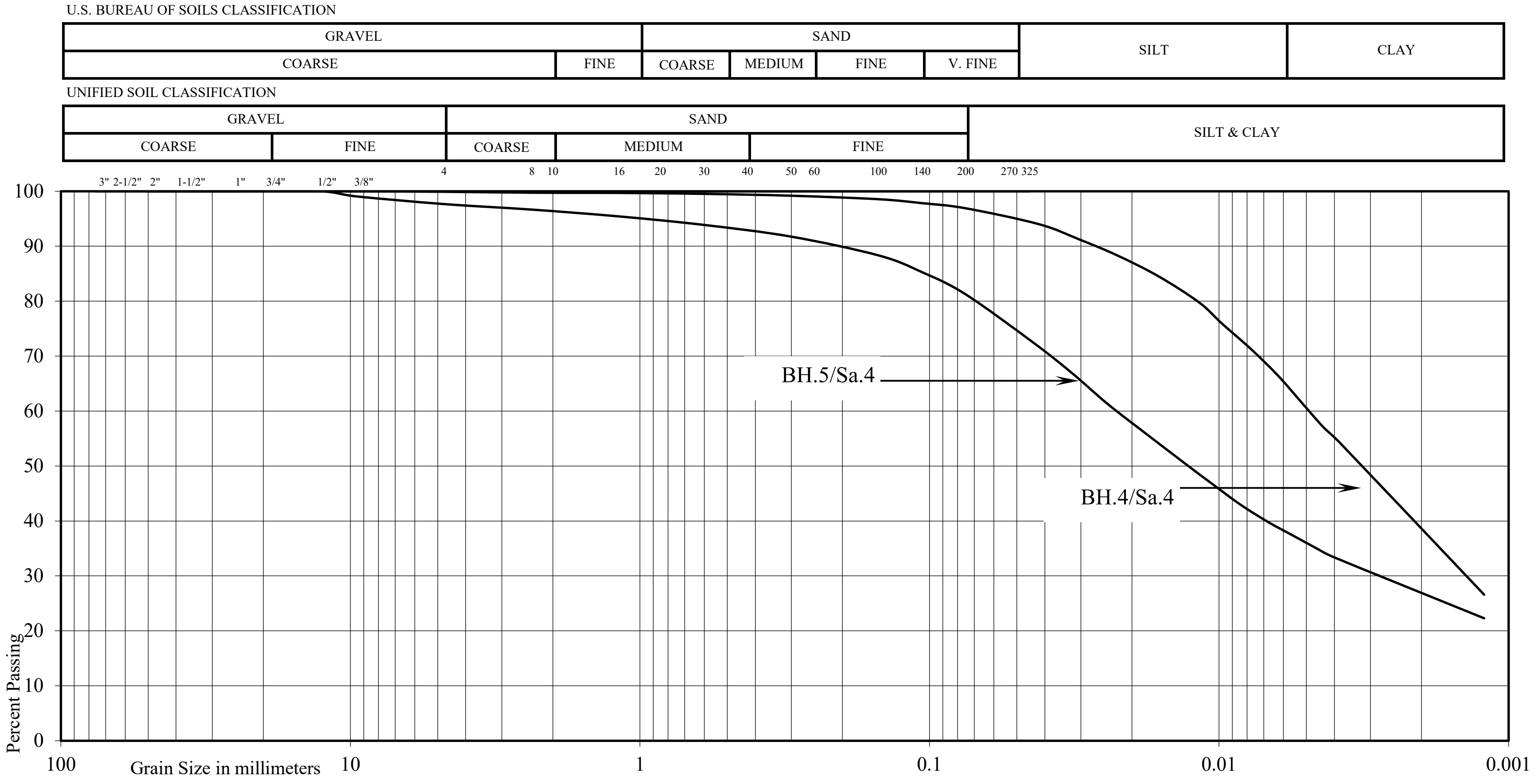
GRAIN SIZE DISTRIBUTION

Reference No: 1905-S182



Project:	Proposed Residential Development	
Location:	13247 and 13233 Nunnville Road, Town of Caledon (Bolton)	
Borehole No:	4	5
Sample No:	4	4
Depth (m):	2.5	2.5
Elevation (m):	243.6	245.7

BH./Sa.	4/4	5/4
Liquid Limit (%) =	40	28
Plastic Limit (%) =	20	17
Plasticity Index (%) =	20	11
Moisture Content (%) =	20	22
Estimated Permeability (cm./sec.) =	10^{-7}	10^{-7}

Classification of Sample [& Group Symbol]:

BH.4/Sa.4 - SILTY CLAY, a trace of sand

BH.5/Sa.4 - SILTY CLAY TILL, some sand, a trace of gravel

Figure: 6

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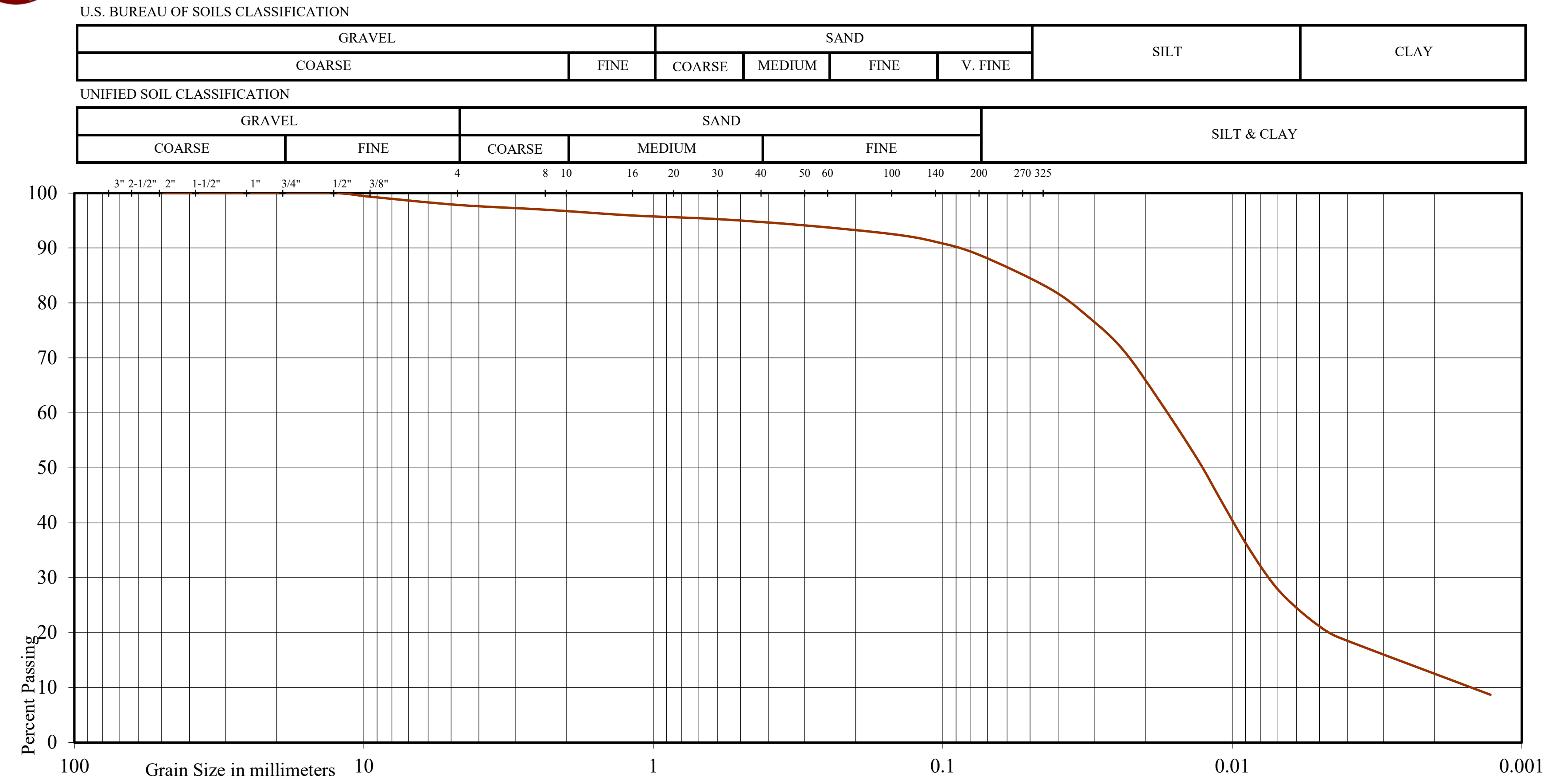
GRAIN SIZE DISTRIBUTION

Reference No: 1905-S182



Project: Proposed Residential Development

Location: 13247 and 13233 Nunnville Road, Town of Caledon (Bolton)

Borehole No: 1

Sample No: 18

Depth (m): 23.0

Elevation (m): 220.3

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

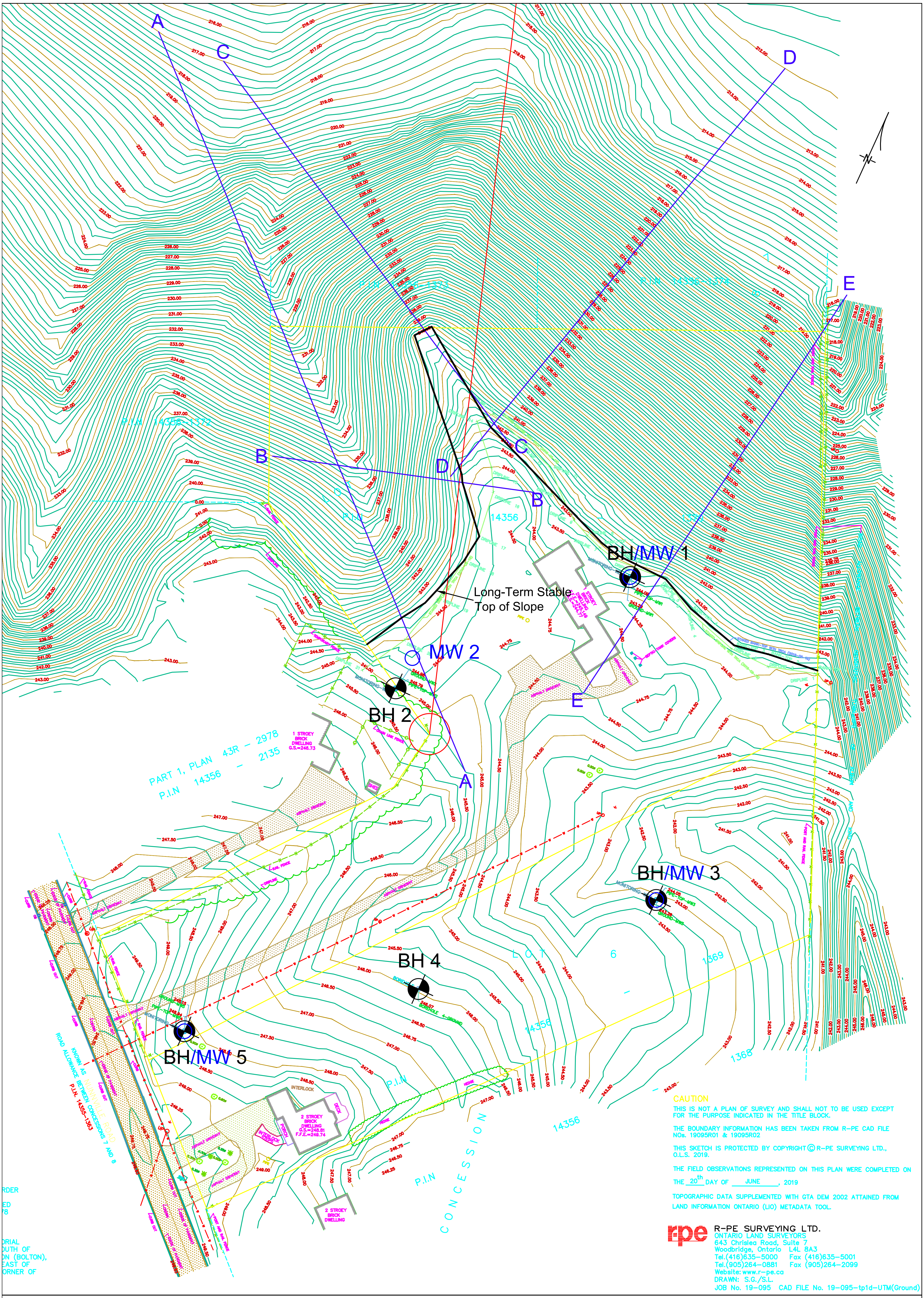
Moisture Content (%) = 22

Estimated Permeability

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

Classification of Sample [& Group Symbol]: SILT, some clay, traces of sand and gravel

Figure: 7



CAUTION

THIS IS NOT A PLAN OF SURVEY AND SHALL NOT TO BE USED EXCEPT FOR THE PURPOSE INDICATED IN THE TITLE BLOCK.

THE BOUNDARY INFORMATION HAS BEEN TAKEN FROM R-PE CAD FILE NOs. 19095R01 & 19095R02

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THE FIELD OBSERVATIONS REPRESENTED ON THIS PLAN WERE COMPLETED ON THE 20th DAY OF JUNE, 2019

TOPOGRAPHIC DATA SUPPLEMENTED WITH GTA DEM 2002 ATTAINED FROM LAND INFORMATION ONTARIO (LIO) METADATA TOOL.

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Website: www.r-pe.ca
DRAWN: S.G./S.L.
JOB No. 19-095 CAD FILE No. 19-095-tp1d-UTM(Ground)

LEGEND

⊕ Borehole

○ Monitoring Well

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

Borehole and Cross-Section Location Plan

SITE: 13247 and 13233 Nunnville Road
Town of Caledon (Bolton)

DESIGNED BY:
CHECKED BY:
DWG NO.: 1
SCALE: 1:1000
REF. NO.: 1905-S182
DATE: July 2019
REV -

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SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

JOB NO.: 1905-S182

REPORT DATE: July 2019

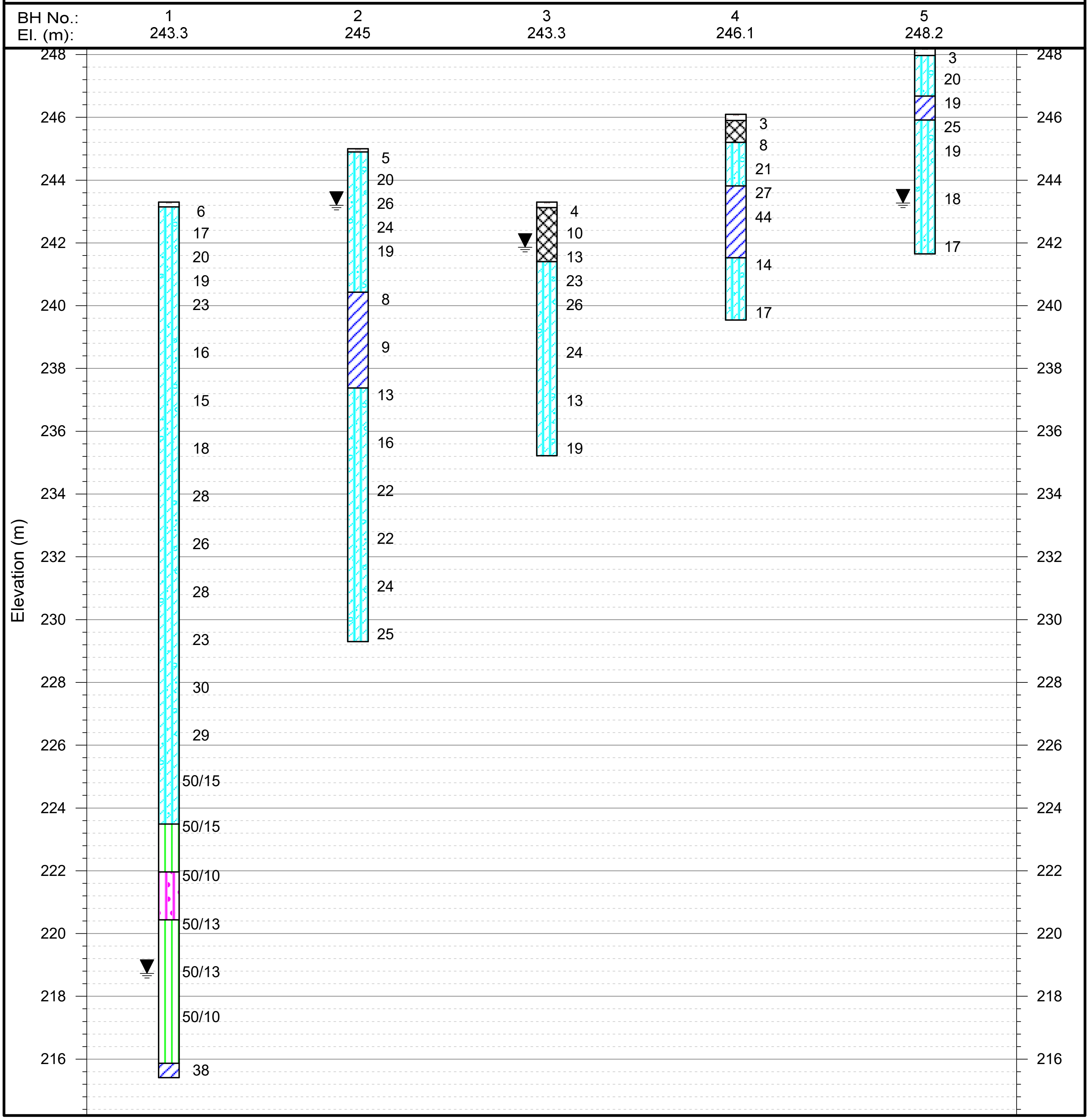
PROJECT DESCRIPTION: Proposed Residential Development

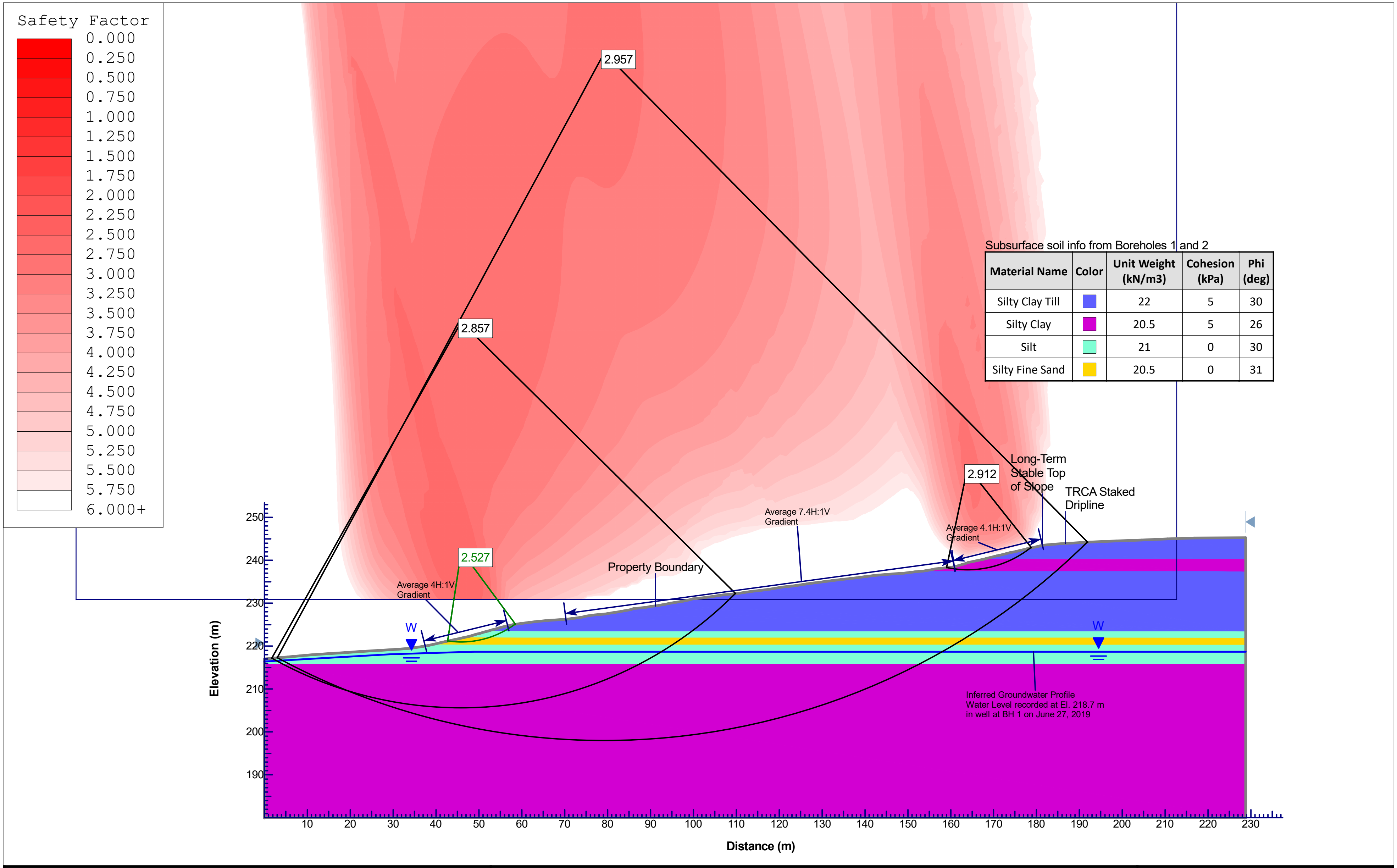
PROJECT LOCATION: 13247 and 13233 Nunnville Road
Town of Caledon (Bolton)

LEGEND

▨	TOPSOIL	▨	SILTY FINE SAND	▨	SILTY CLAY
▩	FILL	▥	SILT	▨	SILTY CLAY TILL

▼ WATER LEVEL (STABILIZED)





Subsurface soil info from Boreholes 1 and 2

Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Silty Clay Till	🟦	22	5	30
Silty Clay	🟪	20.5	5	26
Silt	🟩	21	0	30
Silty Fine Sand	🟨	20.5	0	31

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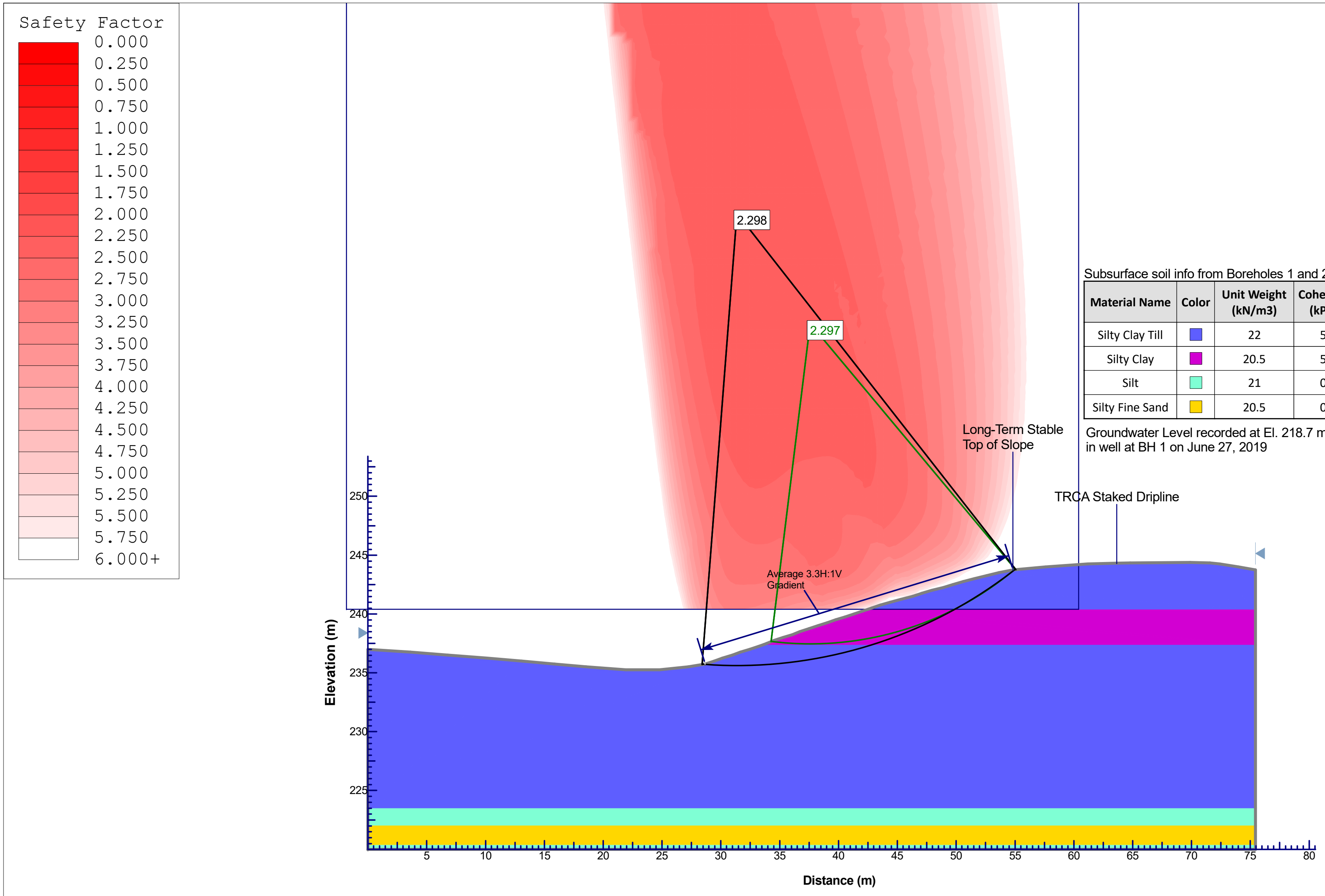
Project Title
Slope Stability Assessment for Proposed Residential Development

Load Case
Cross-Section A-A
(Existing Condition)

Location
13247 and 13233 Nunnville Road, Town of Caledon (Bolton)

Drawn By MM
Checked By BL
Scale 1:1250
Revision
Date July 2019
Reference No. 1905-S182
Drawing No. 3



Subsurface soil info from Boreholes 1 and 2

Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Silty Clay Till	🟦	22	5	30
Silty Clay	🟪	20.5	5	26
Silt	🟩	21	0	30
Silty Fine Sand	🟨	20.5	0	31

Groundwater Level recorded at El. 218.7 m in well at BH 1 on June 27, 2019

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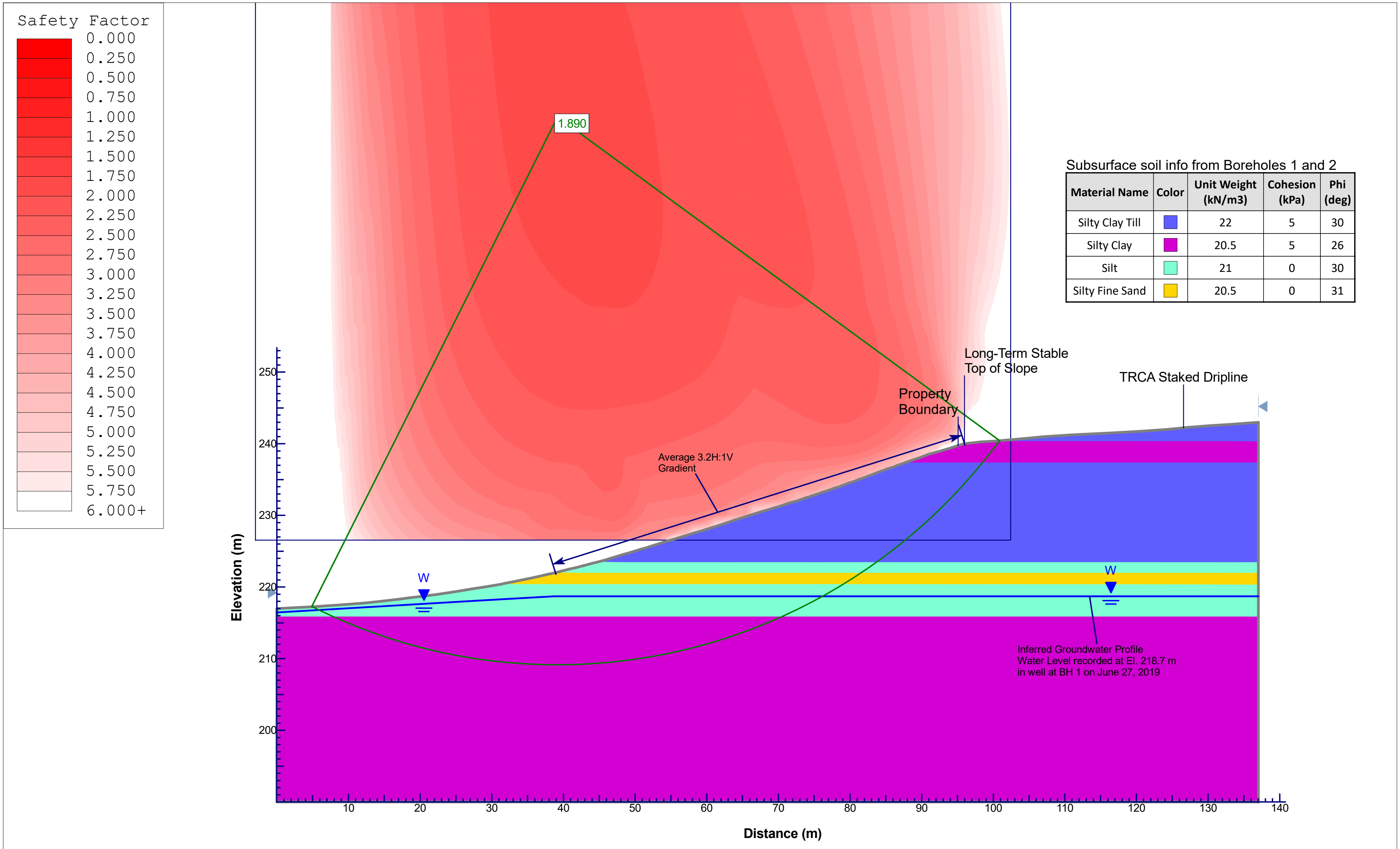
Project Title
Slope Stability Assessment for Proposed Residential Development

Load Case
Cross-Section B-B
(Existing Condition)

Location
13247 and 13233 Nunnville Road, Town of Caledon (Bolton)

Drawn By MM
Checked By BL
Scale 1:500
Revision

Date July 2019
Reference No. 1905-S182
Drawing No. 4



Subsurface soil info from Boreholes 1 and 2

Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Silty Clay Till	🟦	22	5	30
Silty Clay	🟪	20.5	5	26
Silt	🟩	21	0	30
Silty Fine Sand	🟨	20.5	0	31

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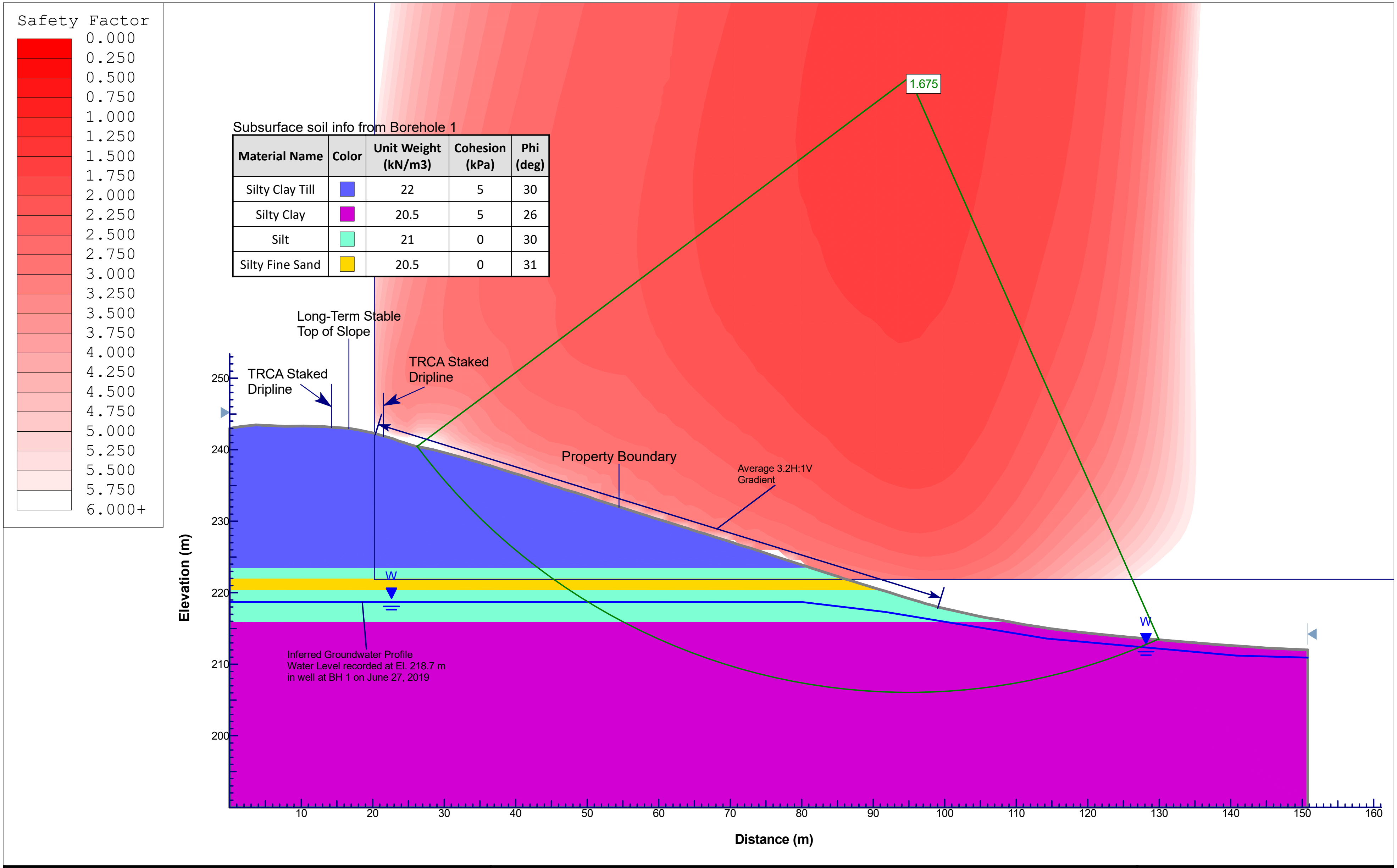
Project Title: Slope Stability Assessment for Proposed Residential Development

Load Case: Cross-Section C-C (Existing Condition)

Location: 13247 and 13233 Nunnville Road, Town of Caledon (Bolton)

Drawn By	MM	Checked By	BL	Scale	1:750	Revision	
Date	July 2019			Reference No.	1905-S182	Drawing No.	5



Subsurface soil info from Borehole 1

Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Silty Clay Till	🟦	22	5	30
Silty Clay	🟪	20.5	5	26
Silt	🟩	21	0	30
Silty Fine Sand	🟨	20.5	0	31

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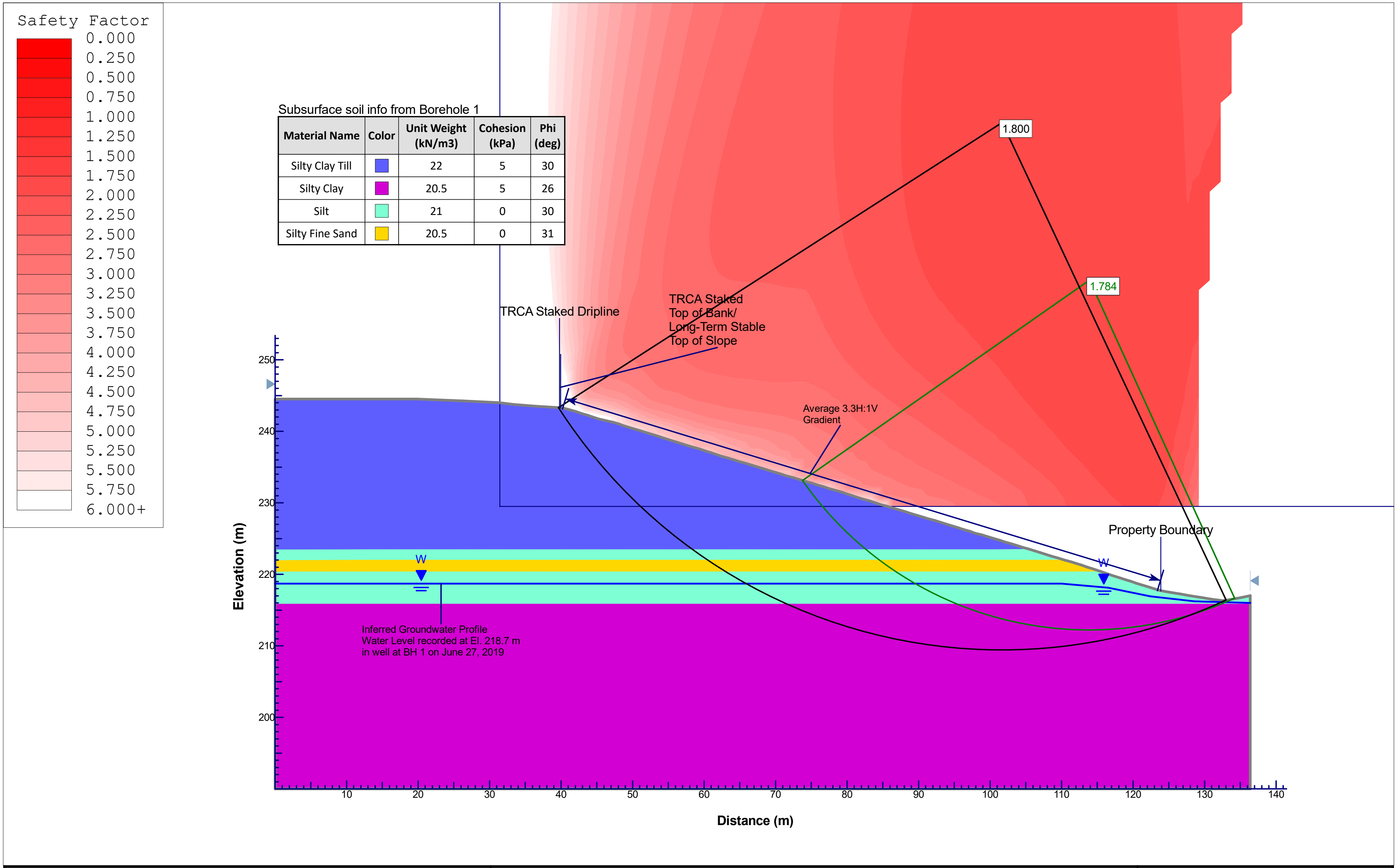
Project Title Slope Stability Assessment for Proposed Residential Development		Load Case Cross-Section D-D (Existing Condition)	
Location 13247 and 13233 Nunnville Road, Town of Caledon (Bolton)			
Drawn By MM	Checked By BL	Scale 1:750	Revision
Date July 2019		Reference No. 1905-S182	Drawing No. 6



Subsurface soil info from Borehole 1

Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Silty Clay Till	🟦	22	5	30
Silty Clay	🟪	20.5	5	26
Silt	🟩	21	0	30
Silty Fine Sand	🟨	20.5	0	31

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Project Title
Slope Stability Assessment for Proposed Residential Development

Load Case
Cross-Section E-E
(Existing Condition)

Location
13247 and 13233 Nunnville Road, Town of Caledon (Bolton)

Drawn By MM
Checked By BL
Scale 1:750
Revision

Date July 2019
Reference No. 1905-S182
Drawing No. 7