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A REPORT TO
BLUE BIRCH PROPERTIES INC.

A GEOTECHNICAL INVESTIGATION
FOR
PROPOSED RESIDENTIAL DEVELOPMENT

PART OF LOT 25, CONCESSION 4
BETWEEN HIGHWAY 26 AND HIDDEN LAKE ROAD,
EAST OF BARCLAY BOULEVARD

TOWN OF THE BLUE MOUNTAINS

REFERENCE NO. 2303-S115

JUNE 2023

DISTRIBUTION

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1.0 INTRODUCTION

In accordance with the written authorization dated March 29, 2023, from Mr. Ron Herczeg of Blue Birch Properties Inc., a geotechnical investigation was carried out for a property with a legal description of Part of Lot 25, Concession 4 in the Town of The Blue Mountains.

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

2.0 SITE AND PROJECT DESCRIPTION

The Town of The Blue Mountains is situated on the south shore of Georgian Bay in the Simcoe Lowlands bordering the Niagara Escarpment where lacustrine sand and silt deposits have bedded onto undulated bedrock groups. Based on the Palaeozoic Geology map for the Collingwood-Nottawasaga region, the site is located on the Whitby Formation, currently known as The Blue Mountain Formation consisting of grey-brown and black shale.

The subject property, encompassing an approximate area of 26.2 acres, is located between Highway 26 and Hidden Lake Road, east of Barclay Boulevard. The central region of the property is occupied by wetlands. The proposed developing areas are located in the northwest corner of the property and in the southern region, west of the existing James Street terminus. At the time of investigation, the site is heavily wooded. The grading is relatively flat across the site, extending until the slope system south of the wetlands which ascends towards Hidden Lake Road.

Based on the Preliminary Concept Plan prepared by Tatham Engineering, the 2 developing areas will consist of low-density residential lots with associated septic bed systems.

3.0 FIELD WORK

The field work, consisting of 4 boreholes extending to depths of 2.7 and 4.3 m, was carried out on April 18, 2023. For groundwater monitoring, 50-mm diameter monitoring wells were installed at the borehole locations. The depth and details of the monitoring wells are shown on the corresponding Borehole Logs. The locations of the boreholes and monitoring wells are shown on Drawing No. 1.

The boreholes were advanced at intervals to the sampling depths by a track-mounted machine equipped with solid-stem augers for soil sampling. Split-spoon samples were recovered for soil classification and laboratory testing. Standard Penetration Tests using the procedures described on the enclosed “List of Abbreviations and Terms” were performed at the sampling depths. The relative density of the non-cohesive strata and the consistency of the cohesive strata are inferred from the ‘N’ values. The field work was supervised and the findings were recorded by a geotechnical technician.

The ground elevation at each borehole location was obtained using handheld Global Navigation Satellite System equipment.

4.0 SUBSURFACE CONDITIONS

Beneath the topsoil veneer, the subsoil profile consists of strata of sand and silty clay, bedding onto grey shale bedrock. Refusal to augering was encountered at the terminated borehole depths of 2.7 and 4.3 m below grade.

Detailed descriptions of the encountered subsurface conditions are presented on the borehole logs, comprising of Figures 1 to 4, inclusive. The stratigraphy is illustrated on the Subsurface Profiles, Drawing No. 2, and the engineering properties of the disclosed soils are discussed herein.

4.1 Topsoil

The revealed topsoil thickness at Boreholes 2, 3 and 4 ranges from 13 to 20 cm. Thicker topsoil may be encountered in areas beyond the borehole locations, especially in local low-lying areas.

4.2 Sand

The investigated areas are deposited with a surficial layer of sand, fine to medium grained with a trace to some silt. Sample examination revealed that the weathered zone extends to depths ranging from 0.8 to 1.6 m below grade. Grain size analyses were performed on 2 sand samples; the gradations are plotted on Figure 5.

The sand is very loose to compact, generally being loose in relative density. The obtained ‘N’ values range from 1 to 29 blows, with a median of 7 blows per 30 cm of penetration.

The obtained natural water content values vary from 6% to 32%, with a median of 21%, indicating that the sand is generally wet.

The engineering properties of the sand are listed below:

- Water erodible material.
- In excavation, the sand will slough to its angle of repose, run with water seepage and boil with a piezometric head of about 0.3 to 0.4 m.

4.3 **Silty Clay**

A layer of silty clay was encountered between the sand and shale bedrock. Sample examination confirmed that the clay is likely a reverted shale deposit, or a transition zone to the underlying shale bedrock. A grain size analysis was performed on a clay sample; the result is plotted on Figure 6.

The obtained 'N' value is 80 blows per 30 cm penetration, indicating that the relative density of the clay is hard.

The Atterberg Limits of the tested clay sample and the natural water content values of all the samples were determined; the results are plotted on the Borehole Logs and summarized below:

Liquid Limit	28%
Plastic Limit	17%
Natural Water Content	14% and 20%

The result indicates that the clay is low in plasticity, and in moist conditions with natural water content values below or near its plastic limit.

The engineering properties of the silty clay are listed below:

- Highly frost susceptible and high in soil-adfreezing potential.
- Low water erodibility.
- The clay will be stable in relatively steep excavation; however, if remained open for an extended period of time, localized sloughing may occur.

4.4 Shale Bedrock

Weathered shale bedrock was encountered in all boreholes at shallow depths of 1.8 and 2.3 m below grade, or between El. 180.1 m in the north and El. 182.2 m in the south. Refusal to augering was encountered within the shale bedrock at depths of 2.7 and 4.3 m below grade, or between El. 179.7 m in the north and El. 180.3 m in the south.

The shale bedrock, part of the Blue Mountain Formation, is grey in colour and is a laminated, sedimentary, moderately soft rock composed predominantly of clay material. The upper layer of the shale is often fissured as a result of the weathering process; the weathered condition often extends to about 2 to 3+ m below the surface of the bedrock. Infiltrated precipitation and groundwater from the overburden soils will often permeate the fissures in the rock and, in places, will be under subterranean artesian pressure. However, due to the low permeability of the clayey soil, the shale bedrock is considered as a poor aquifer, and the groundwater yield from the rock will be limited.

The shale is susceptible to disintegration and swelling upon exposure to air and water, with subsequent reversion to a clayey soil, but any laminated limy layers would remain as rock slabs.

The weathered rock can be excavated with considerable effort by a heavy-duty backhoe equipped with a rock-ripper; however, excavation will become progressively more difficult with depth into the sound shale. Efficient removal of the sound shale may require the aid of pneumatic hammering. When excavating the sound shale, slight lateral displacement of the excavation walls is often experienced. This is due to the release of residual stress stored in the bedrock mantle and the swelling characteristic of the rock. The excavated spoil will contain a large amount of hard limy and sandy rock slabs, rendering it virtually impossible to obtain uniform compaction. Therefore, unless the spoil is sorted, it is considered unsuitable for engineering applications.

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 +
Sand	6 to 32 (median 21)	8 to 9	6 to 12
Silty Clay	14 and 20	17	14 to 20

The on-site soils are generally suitable for structural backfill, provided that they are sorted free of organics and topsoil inclusions. The sand is too wet and will require aeration by stockpiling prior to structural compaction.

The lifts for compaction should be limited to 20 cm, or to a suitable thickness assessed by test strips performed by the compaction equipment. If an appreciable amount of shale fragments is mixed with the backfill material, it must either be sorted or must not be used for structural backfill and/or construction of engineered fill.

5.0 GROUNDWATER CONDITION

Monitoring wells were installed in all of the borehole locations. Monthly groundwater levels were subsequently recorded from the wells in April, May and June, 2023, and are summarized in Table 2.

Table 2 - Groundwater Levels

Borehole/ Monitoring Well No.	Ground El. (m)	Well Depth (m)	Measured Groundwater Levels					
			April 25, 2023		May 25, 2023		June 16, 2023	
			Depth (m)	El. (m)	Depth (m)	El. (m)	Depth (m)	El. (m)
1	182.5	2.7	1.17	181.31	1.29	181.19	1.53	180.95
2	182.4	2.7	0.87	181.51	1.05	181.33	1.22	181.16
3	183.0	2.7	0.28	182.67	0.58	182.95	0.86	182.09
4	184.5	4.3	0.87	183.58	1.29	183.16	2.08	182.37

The recorded levels are highest in April, ranging from 0.28 to 1.17 m below grade, or at El. 181.31 and 181.51 in the north, and at El. 182.67 and 183.58 m in the south, showing a general northerly drainage trend following the topography. The groundwater is subject to seasonal fluctuations.

6.0 SLOPE STABILITY REVIEW

The southern limit of the property is bordered with a slope that ascends towards Hidden Lake Road. The proposed southern lots lie at the existing bottom of slope. In order to establish the Long-Term Stable Toe of Slope (LTSTOS), a desktop slope stability review was carried out, combined with a visual inspection of the existing slope.

Based on the topographic contour plan provided by Tatham Engineering, dated June 2023, the slope has an approximate height of 26 to 35 m, measured from the tableland at the neighbouring properties fronting Hidden Lake Road to the existing toe of slope. The slope gradients range from 1V:1.4 to 4.4+ H, with a widening plateau observed between the upper and lower portions of slope towards the east. There is no watercourse at the bottom of slope.

Visual inspection revealed that the slope is well vegetated with trees, majority of which are in an upright orientation. No sign of deep-seated failure was observed. The slope face is generally covered with leaves, except around oversteepened areas where the slope face becomes bare. Drainage gullies were observed in areas, suggesting periodic concentrated surface runoff from the top of slope.

Borehole information is not available for the overall slope as drilling would otherwise be required to take place on the public road or in the neighbours' properties. Alternatively, a generally accepted stable gradient of 1V:3H is projected from the top of slope down towards the bottom of bank to mark the LTSTOS. Where the projection lands on the plateau, a secondary projection is introduced from the top of the lower slope. These extrapolations are demonstrated on Cross-Sections A-A and B-B, shown on Drawing Nos. 3 and 4. The locations of the cross-sections, along with the established LTSTOS based on the stable gradient setback are presented on Drawing No. 1. The surface profile is interpreted from the contours on the aforementioned topographic plan.

Any development setback buffer behind the LTSTOS will be subject to the discretion of the Grey Sauble Conservation Authority (GSCA).

Upon completion of the construction, the following geotechnical constraints should be stipulated in order to prevent the disturbance of the existing slope:

1. The prevailing vegetative cover must be maintained, since its extraction would deprive the bank of the rooting system that is reinforcement against soil erosion by weathering. If for any reason the vegetation cover is stripped during construction, it must be reinstated to its original, or better than its original, protective condition.
2. The topsoil and leaf cover on the bank face should not be disturbed, since this provides insulation and screen against frost wedging and rainwash erosion.
3. Dumping of loose fill on the slope should be prohibited.

Although the top of slope resides within neighbouring properties, grading activities or landscaping features which may cause ponding or saturation of the crown of the bank and concentrated runoff should be discouraged. Any slope failures observed should immediately be communicated with the neighbours and conservation authority to prevent further destabilization of the slope.

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

7.0 DISCUSSION AND RECOMMENDATIONS

Beneath the topsoil veneer, the subsoil profile consists of strata of generally loose sand and hard silty clay, bedding onto grey shale bedrock at shallow depths of 1.8 and 2.3 m below grade, or between El. 180.1 m in the north and El. 182.2 m in the south. Refusal to augering was encountered at the terminated borehole depths of 2.7 and 4.3 m below grade. The surficial weathered zone extends to depths of 0.8 to 1.6 m below grade.

The highest groundwater levels recorded from the monitoring wells range from 0.28 to 1.17 m below grade, or at El. 181.31 and 181.51 in the north, and at El. 182.67 and 183.58 m in the south. The groundwater is subject to seasonal fluctuations.

It is understood that both the north and south developing areas of the property will consist of low-density residential lots with septic bed systems. However, detailed site plan was not available for review at the time of the report preparation. The following geotechnical considerations warrant special attention:

1. The topsoil must be stripped for development; it can be reused for general landscaping purposes only. Any surplus should be removed off site.
2. The weathered soil should be inspected prior to any placement of earth fill for site grading purpose. Where required, the weathered soil should be subexcavated, sorted free of any organic, topsoil, and/or other deleterious material, before reusing for structural backfill.

3. Where additional fill is required for site grading, the earth fill can be placed in an engineered manner for conventional footing and road construction.
4. The engineered fill, sound native soils and shale bedrock are suitable for supporting structures founded on conventional spread and strip footings.
5. In view of the wet sand deposit in the upper stratigraphy and relatively shallow groundwater measurements, it is recommended that any basement floor be founded at least 1.0 m above the seasonal high groundwater level. Otherwise, waterproofing of basements should be implemented and the structures should be supported using raft foundation in order to resist the hydrostatic pressure.
6. The yield of groundwater from the sand deposit will likely be moderate to appreciable, and persistent. Excavation extending into the saturated sand deposit will require construction dewatering.

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, and the assessment given herein is general in nature based on the borehole findings. Should this become apparent during construction, a geotechnical engineer must be consulted to determine whether the following recommendations require revision.

7.1 Site Preparation

The topsoil and vegetation at the ground surface must be removed for development. Where additional fill is required for site grading, the earth fill can be placed in an engineered manner for conventional footing construction, site services support and road construction. The engineering requirements for a certifiable fill are presented below:

1. The subgrade must be inspected and proof-rolled prior to any fill placement. Badly weathered soils should also be subexcavated, sorted free of topsoil inclusions and deleterious materials, if any, aerated and properly compacted in layers.
2. Inorganic soils must be used for the fill, and they must be uniformly compacted in lifts of 20 cm thick 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 or 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 being hauled to the site.
5. The fill operation must be inspected on a full-time basis by a technician under direction of a geotechnical engineer.
6. The engineered fill should not be placed during period when 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.
7. 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.
8. The foundations 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.
9. Despite stringent control in the placement of the engineered fill, variations in soil type and density may occur in the engineered fill. Therefore, the foundations must be properly reinforced, or be designed by the structural engineer for the project. The total and differential settlements of 25 mm and 20 mm, respectively, should be considered in the design of the foundation founded on engineered fill.
10. 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.

7.2 **Foundation**

At the time of report preparation, detail design of the development is not available for review. For conventional strip and spread house footings founded on engineered fill, the recommended bearing pressures are as follows:

- Maximum Bearing Pressure at Serviceability Limit State (SLS) = 100 kPa
- Factored Ultimate Bearing Pressure at Ultimate Limit State (ULS) = 150 kPa

Based on the borehole information, the bearing pressures recommended for conventional footings founded onto sound native soils or shale bedrock below the weathered soils are presented in Table 3.

Table 3 - Founding Levels

BH No.	Recommended Maximum Allowable Soil Pressure and Corresponding Founding Level			
	100 kPa (SLS) 150 kPa (ULS)		300 kPa (SLS) 450 kPa (ULS)	
	Depth (m)	El. (m)	Depth (m)	El. (m)
1	1.0 or +	181.5 or -	2.3 or +	180.2 or -
2	1.0 or +	181.4 or -	2.0 or +	180.4 or -
3	-	-	1.6 or +	181.4 or -
4	-	-	1.5 or +	183.0 or -

Where basements are contemplated, extending below the groundwater or into saturated sand, waterproofing of the basements should be implemented and the raft foundation can be designed with bearing pressures of 100 kPa (SLS) and 150 kPa (ULS) at a founding depth of 1.5 m below the prevailing ground surface. These measures can be further discussed once the detailed design of the building is available for review.

The total and differential settlements of footing designed for the recommended bearing pressure at SLS are estimated at 25 mm and 20 mm, respectively. For footings founded onto shale bedrock, the settlements will be negligible.

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

Footings exposed to weathering, or in unheated areas, should have at least 1.4 m of earth cover for protection against frost action.

Where the footing subgrade consists of wet sand or shale bedrock which is subject to disintegration and swelling after it is exposed, a concrete mud-slab of lean mix concrete, 8 to 10 cm in thickness, should be poured immediately after subgrade preparation and inspection to protect the approved subgrade.

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

7.3 Underground Structure

Where underground structures are proposed, they should be designed for the lateral earth pressure using the soil parameters provided in Table 6.

Based on the borehole findings, it is recommended that the basement floor be maintained at least 1.0 m above the seasonal high groundwater level. The perimeter walls of the basement structure should be damp-proofed and provided with perimeter subdrains at the wall base. Backfill of the open excavation should consist of free-draining granular material (Drawing No. 5) unless prefabricated drainage board is installed over the entire wall below grade. The subdrains, connected to a positive outlet, should be encased in a fabric filter to protect them against blockage by silting.

The subgrade of the basement slab must consist of sound native soil or well compacted inorganic earth fill or engineered fill. The subgrade should be inspected and assessed by proof-rolling prior to slab-on-grade construction. Where loose or soft subgrade is detected, it should be subexcavated and replaced with inorganic material, compacted to at least 98% SPDD.

The concrete slab should be constructed on a minimum 15 cm thick granular base, consisting of 19-mm CRL, or equivalent, compacted to its maximum SPDD.

Should the basement structure be founded less than 1.0 m above the seasonal high groundwater level, the basement structure should be waterproofed and designed for hydrostatic uplift pressure. Further discussion can be provided once detailed design is available for review.

7.4 Underground Services

It is our understanding that the site will be serviced with watermain connection only. No municipal storm or sanitary sewers is available. Water pipe connections are generally shallow, and therefore installation of service in shale is not anticipated.

The subgrade for underground services should consist of sound native soils or properly compacted earth fill. Where soft or loose soil is encountered at the invert level, it must be subexcavated and replaced with properly compacted bedding material.

A Class 'B' bedding is recommended for construction of the underground services. The bedding material should consist of compacted 19-mm CRL, or equivalent, compacted to at least 98% SPDD. In the saturated sand deposit, a Class 'A' bedding can be considered for proper pipe support.

The pipe joints should be leak-proof or wrapped with an appropriate waterproof membrane to prevent migration of fines due to leakage, leading to a loss of subgrade support and subsequent pipe collapse. Openings to subdrains should be shielded by a fabric filter to prevent silting.

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

The service pipes and metal fittings should be protected against corrosion. For estimation of anode weight requirements, the electrical resistivities of the disclosed soils presented in Table 6 can be used. The proposed anode weight must meet the minimum requirements as specified by the Town of The Blue Mountains.

7.5 Backfilling Trenches and Excavated Areas

The on-site inorganic soils are suitable for trench backfill. Wet soils will require aeration prior to its use as structural backfill. Where necessary, shale bedrock should be broken into pieces less than 15 cm in diameter prior to be reused for backfill.

The backfill material should be compacted to at least 95% SPDD. In areas below the slab-on-grade and in the zone within 1.0 m below the pavement subgrade, the backfill should be compacted to at least 98% SPDD with a moisture content 2% to 3% drier than the optimum.

This is to provide the required stiffness for floor or pavement construction. The lift of each backfill layer should be limited to a thickness of 20 cm, or the thickness should be determined by test strips at the time of compaction.

In normal construction practice, the problem areas of pavement settlement largely occur adjacent to foundation walls, columns, manholes, catch basins and services crossings. In areas which are inaccessible to a heavy compactor, sand backfill which can be appropriately compacted using a smaller vibratory compactor should be used.

7.6 Pavement Design

The access roads to the southern lots via James Street and to the northern lots via Railway Street/Barclay Boulevard are currently non-paved gravel roads. Should the future access routes be maintained as gravel roads/cul-de-sacs, the recommended roadway makeup is presented in Table 4. If the existing granular make up of James Street and Railway Street/Barclay Boulevard can be identified, the new road sections should be constructed using similar thicknesses or have a transition zone to taper the granular layers to avoid abrupt change in the pavement structure.

Table 4 - Granular Pavement Design

Course	Thickness (mm)	OPS Specifications
Granular Base	150	Granular 'A' or equivalent
Granular Sub-base	300	Granular 'B' or equivalent

For future paved roads, the recommended pavement design for local residential road, meeting the minimum specification from the Town of The Blue Mountain, is presented in Table 5.

Table 5 - Pavement Design

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	50	HL3
Asphalt Binder	50	HL4 or HL8
Granular Base	150	Granular 'A' or equivalent
Granular Sub-base	450	Granular 'B' or equivalent

In preparation of the pavement subgrade, all topsoil and compressible material should be removed. The subgrade should be proof-rolled and inspected. Any soft spots identified must be subexcavated and replaced with inorganic earth fill. The subgrade within 1.0 m below the underside of the granular sub-base must be compacted to at least 98% SPDD, with a water content at 2% to 3% drier than the optimum. All the granular bases should be compacted to 100% SPDD.

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

- The pavement subgrade should be properly crowned and smooth-rolled to allow interim precipitation to be properly drained.
- Lot areas adjacent to the pavement should be properly graded to prevent ponding of large amounts of water.
- In extreme cases during the wet seasons, if soft or weak subgrade is identified, it can be replaced by compacted granular material to compensate for the inadequate strength of the soft or weak subgrade. This can be assessed during construction.
- Subdrains consisting of filter-sleeved weepers should be provided along the driveway and in the lower spots where surface water may accumulate, and be connected to a positive outlet.

7.7 <u>Septic System</u>

It is understood that septic systems will be provided for each of the house lots.

The limitations for normal in-ground septic tile bed construction are that the bottom of the absorption trenches, or the surface of a filter medium, be located a minimum of 0.9 m above the highest groundwater level and above soils with a percolation time (‘T’) not exceeding 50 min/cm. Based on the borehole findings, the site is underlain predominantly by strata of fine to medium sand and silty clay. The associated percolation time can be referred to Table 6. Although the native sandy soil is relatively high in permeability and is suitable for in-ground septic tile bed construction, but considering the high groundwater table on site, the proposed septic beds will likely have to be raised.

The soil in the treatment zone should possess acceptable effluent absorption properties expressed in a percolation time of between 10 and 20 min/cm. The tile bed should consist of at least medium or high permeable material. Detailed design of the septic tile bed system can be obtained from the latest Ontario Building Code.

In order to enhance an efficient bed operation, the following requirements should be incorporated into the septic tile bed construction:

- The bed should be located in an unshaded area.
- All topsoil and organics should be stripped from the tile bed area.
- For the raised septic tile bed, the sand filter should be keyed into the soil mantle to about 15 cm from the transition depth.
- Grading of the surrounding areas should be such that it directs surface runoff away from the tile bed area. In the low-lying areas, the septic tile bed should be elevated so that surface runoff will not pond.
- The fissured pattern of the underlying soils should not be disturbed, as this would reduce their capacity for in-ground effluent absorption.

The recommendations presented above are subject to the approval of the local regulatory agency.

Given that the septic system will likely be raised and constructed on engineered fill, it is not possible to provide an estimate on the coefficient of permeability until the engineered fill is completed. If the actual coefficient of permeability, percolation time and infiltration rate are required, it is recommended that an in-situ percolation test be performed at the designed location at the proposed invert of the septic tile bed prior to the design and/or construction of the tile bed system.

7.8 Soil Parameters

The recommended soil parameters for the project design are given in Table 6.

Table 6 - Soil Parameters

Unit Weight and Bulk Factor	**Unit Weight (kN/m^3)**		**Estimated Bulk Factor**	
	Bulk	**Submerged**	**Loose**	**Compacted**
Silty Clay	20.5	10.5	1.33	1.03
Sand	20.0	10.0	1.25	1.00

Lateral Earth Pressure Coefficients	**Active K_a**	**At Rest K_o**	**Passive K_p**
Compacted Earth Fill	0.40	0.55	2.50
Silty Clay	0.36	0.53	2.77
Sand	0.32	0.48	3.12

Table 6 - Soil Parameters Cont'd

Estimated Coefficient of Permeability (K) and Percolation Time (T)	**K (cm/sec)**	**T (min/cm)**
Silty Clay	10^{-7}	80+
Sand	10^{-3}	8
Estimated Electrical Resistivity		**(ohm·cm)**
Silty Clay		3500
Sand		6000
Coefficients of Friction		
Between Concrete and Granular Base		0.50
Between Concrete and Native Soils or Compacted Earth Fill		0.35

7.9 Excavation

Excavation should be carried out in accordance with Ontario Regulation 213/91. The types of soils to be excavated are classified in Table 7.

Table 7 - Classification of Soils for Excavation

Material	Type
Weathered Shale	1
Hard Silty Clay	2
Weathered Soils and Sand (above groundwater)	3
Saturated Soils	4

In excavation, the yield of groundwater from the wet sand deposit may be moderate to appreciable, and persistent. In order to provide a stable subgrade for the services or foundation construction, the groundwater should be depressed to at least 0.5 m below the intended bottom of excavation. Therefore, where excavation extends below the groundwater into the saturated deposit, more extensive construction dewatering will likely be required. The appropriate dewatering method can be assessed by test pumping at the site.

Prospective contractors should 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 prior to excavating. These test pits should be allowed to remain open for a period of time to assess the trenching conditions.

8.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of Blue Birch Properties Inc. and for review by its designated consultants, contractors and government agencies. The material in the report reflects the judgement of Hui Wing Yang, P.Eng. and Kin Fung Li, P.Eng., in light of the information available to it at the time of preparation.

Use of the report is subject to the conditions and limitations of the contractual agreement. 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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HWY/KFL

LICENSED PROFESSIONAL ENGINEER
June 29, 2025
H. W. YANG
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PROVINCE OF ONTARIO

LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

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

SAMPLE TYPES

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

PENETRATION RESISTANCE

Dynamic Cone Penetration Resistance:

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

Plotted as '—•—'

Standard Penetration Resistance or 'N' Value:

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

Plotted as '○'

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

SOIL DESCRIPTION

Cohesionless Soils:

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

Cohesive Soils:

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

Method of Determination of Undrained Shear Strength of Cohesive Soils:

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

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

METRIC CONVERSION FACTORS

1 ft = 0.3048 metres

1lb = 0.454 kg

1 inch = 25.4 mm

1ksf = 47.88 kPa

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JOB NO.: 2303-S115

LOG OF BOREHOLE: 1

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Solid Stem Augers

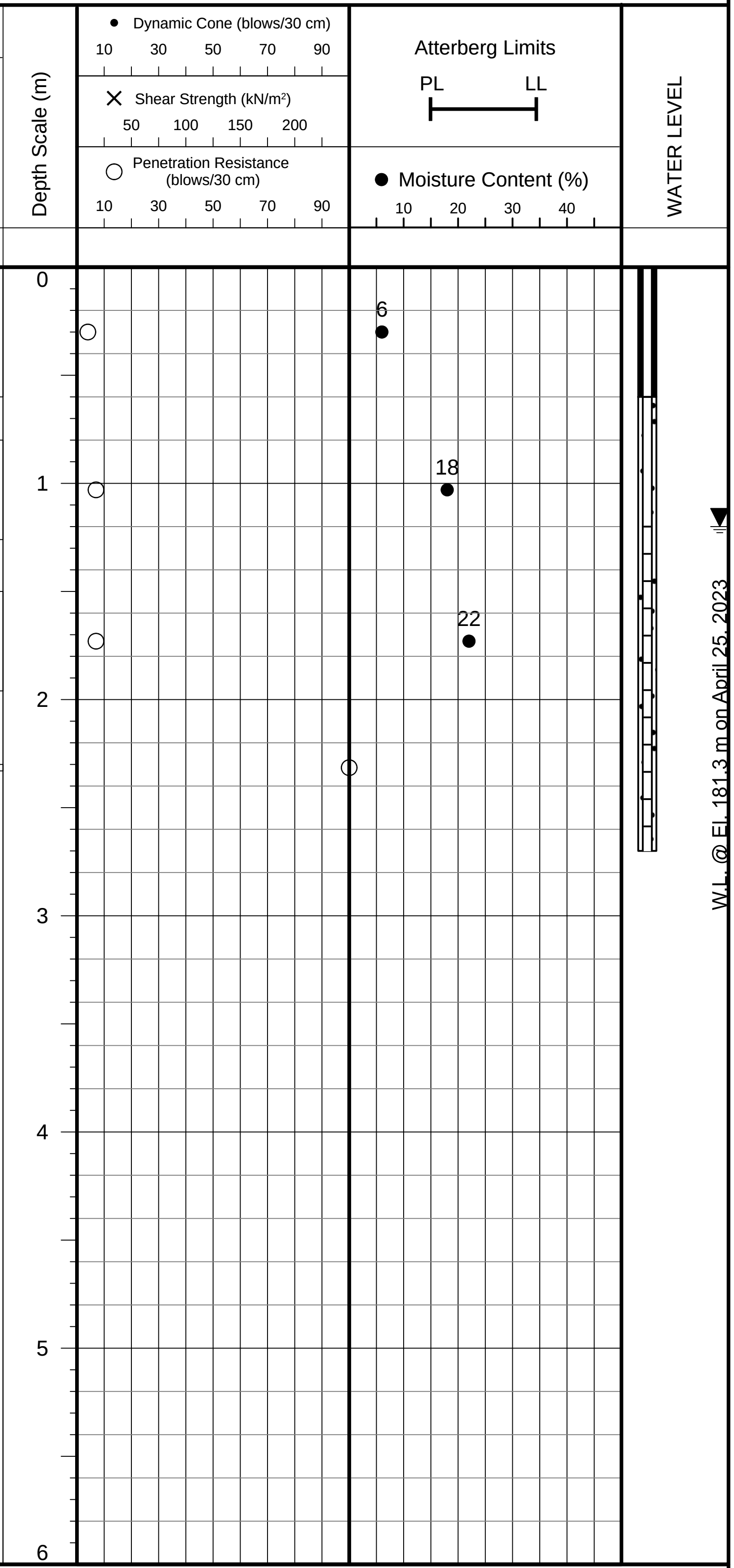
PROJECT LOCATION: Part of Lot 25, Conc. 4, B/W Hwy 26 & Hidden Lake Rd.,
East of Barclay Blvd., Town of The Blue Mountains

DRILLING DATE: April 18, 2023

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
182.5	Ground Surface			
0.0	Brown, very loose to loose **FINE TO MEDIUM SAND** weathered a trace of silt becoming wet below 1.0 m	1 2 3	DO DO DO	4 7 7
180.2 2.3	Grey, weathered **SHALE BEDROCK**	4	DO	70/3
179.8 2.7	occ. rock fragments			

END OF BOREHOLE
Refusal on Augers

Installed 50-mm Ø PVC monitoring well to
2.7 m, completed with 1.5 m screen
Sand backfill from 0.6 to 2.7 m
Bentonite seal from 0.0 to 0.6 m
Provided with a steel monument casing



JOB NO.: 2303-S115

LOG OF BOREHOLE: 2

FIGURE NO.: 2

PROJECT DESCRIPTION: Proposed Residential Development

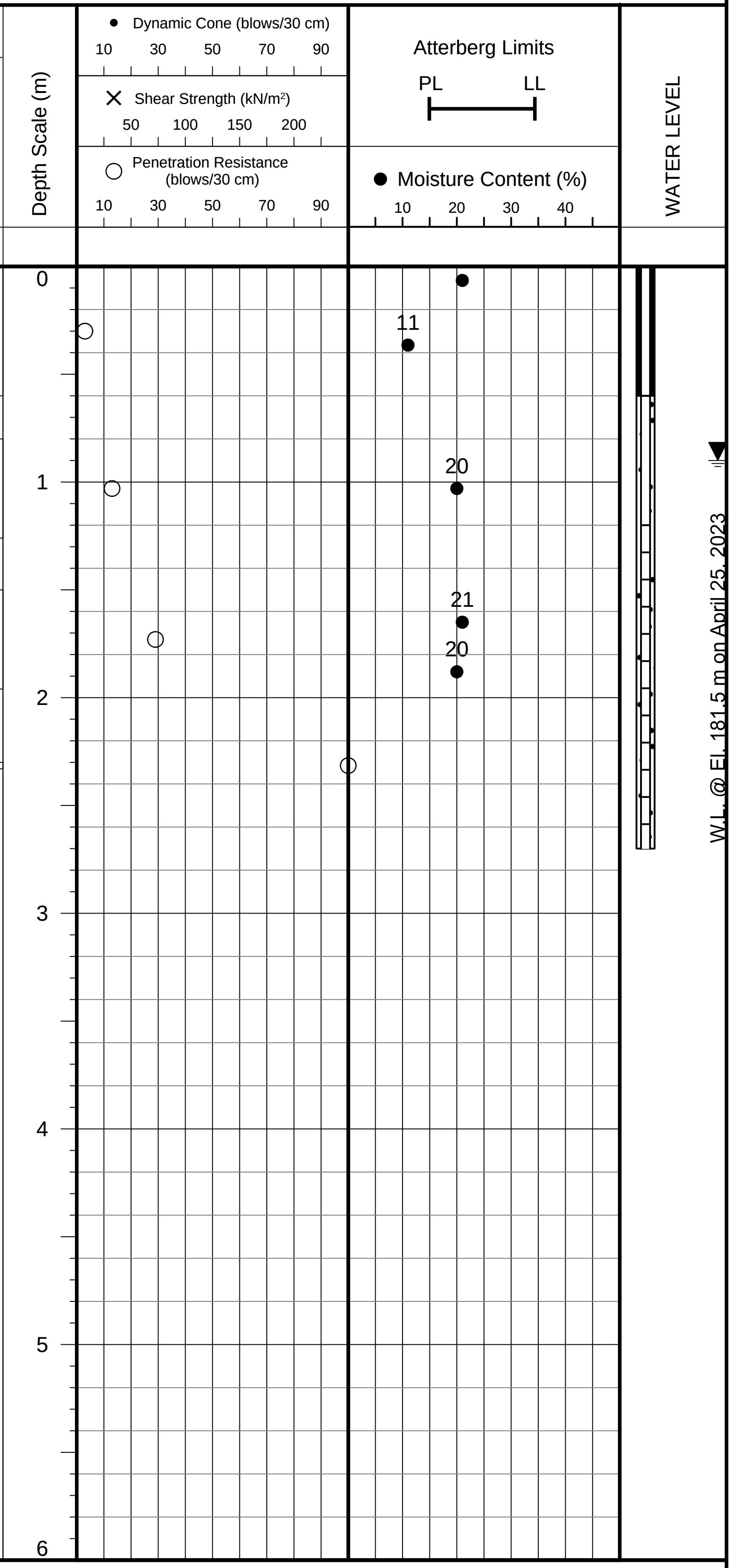
METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: Part of Lot 25, Conc. 4, B/W Hwy 26 & Hidden Lake Rd.,
East of Barclay Blvd., Town of The Blue Mountains

DRILLING DATE: April 18, 2023

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
182.4	Ground Surface			
0.0	**13 cm TOPSOIL**	1A	DO	3
	Very loose to compact **FINE TO MEDIUM SAND** weathered, trace organics	1B		
	a trace of silt becoming wet below 1.0 m	2	DO	13
	brown changing to grey	3A	DO	29
180.6 1.8	Grey **SILTY CLAY** transitioning to shale bedrock	3B		
180.1 2.3	Grey, weathered **SHALE BEDROCK**	4	DO	70/3
179.7 2.7	**END OF BOREHOLE** **Refusal on Augers**			

Installed 50-mm Ø PVC monitoring well to
2.7 m, completed with 1.5 m screen
Sand backfill from 0.6 to 2.7 m
Bentonite seal from 0.0 to 0.6 m
Provided with a steel monument casing



JOB NO.: 2303-S115

LOG OF BOREHOLE: 3

FIGURE NO.: 3

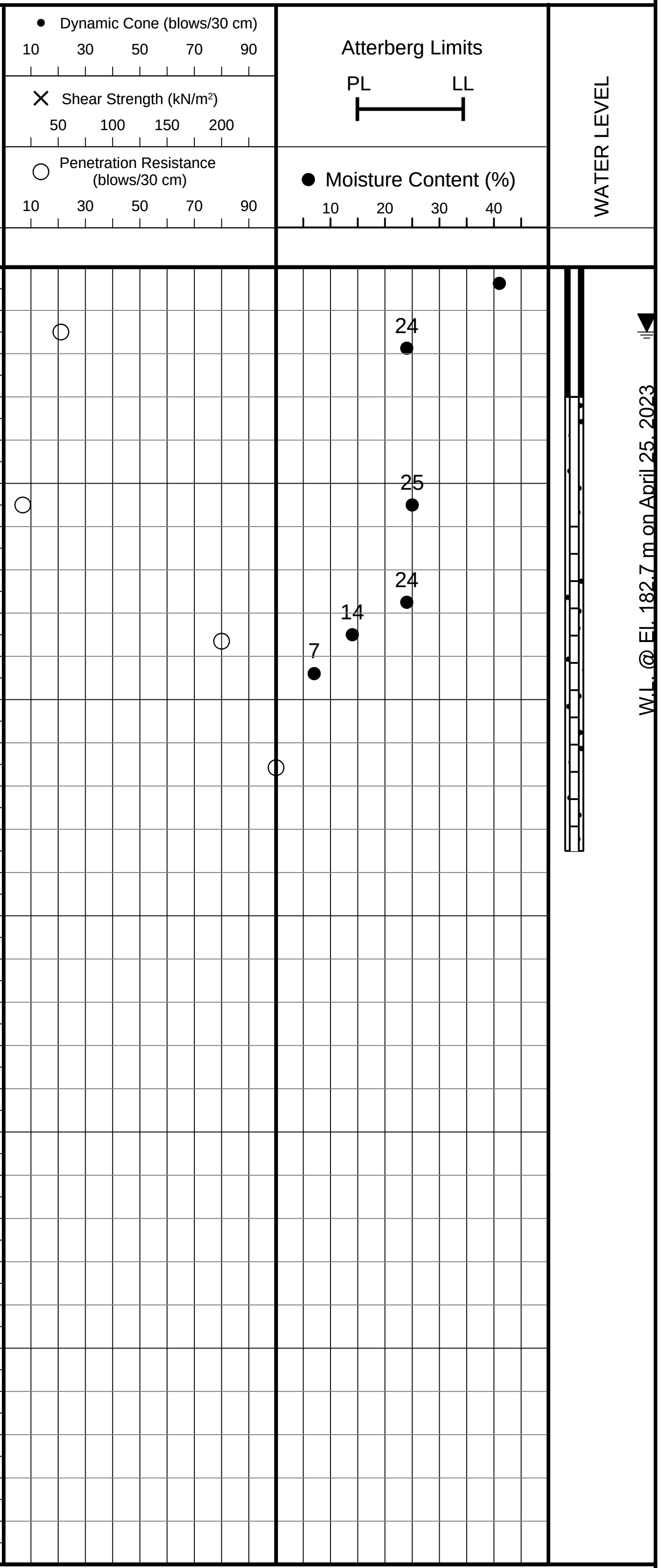
PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: Part of Lot 25, Conc. 4, B/W Hwy 26 & Hidden Lake Rd.,
East of Barclay Blvd., Town of The Blue Mountains

DRILLING DATE: April 18, 2023

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value	Depth Scale (m)
183.0	Ground Surface				
0.0	**15 cm TOPSOIL**	1A	DO	21	0
	Brown, loose, weathered, wet obstruction: tree root **FINE TO MEDIUM SAND**	1B			
	a trace to some silt traces of coarse sand and gravel	2	DO	7	1
181.4 1.6	Grey, hard **SILTY CLAY**	3A 3B	DO	80	
181.2 1.8	(reverted shale)	3C			
	Grey, weathered **SHALE BEDROCK**	4	DO	70/3	2
180.3 2.7	**END OF BOREHOLE** **Refusal on Augers**				3
	Installed 50-mm Ø PVC monitoring well to 2.7 m, completed with 1.5 m screen Sand backfill from 0.6 to 2.7 m Bentonite seal from 0.0 to 0.6 m Provided with a steel monument casing				4 5 6



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JOB NO.: 2303-S115

LOG OF BOREHOLE: 4

FIGURE NO.: 4

PROJECT DESCRIPTION: Proposed Residential Development

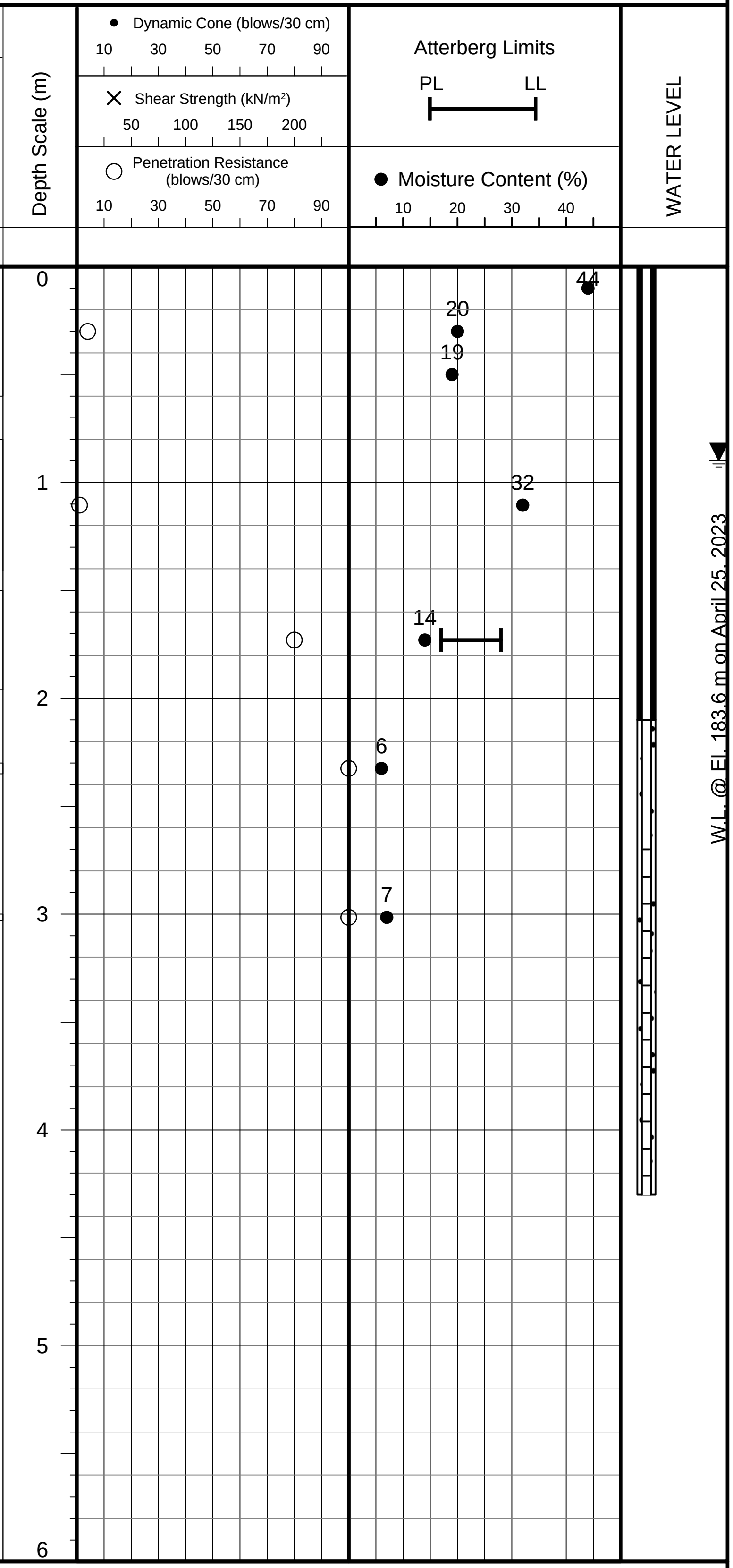
METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: Part of Lot 25, Conc. 4, B/W Hwy 26 & Hidden Lake Rd.,
East of Barclay Blvd., Town of The Blue Mountains

DRILLING DATE: April 18, 2023

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
184.5	Ground Surface			
0.0	**20 cm TOPSOIL**	1A	DO	4
	Dark-brown/brown, very loose, weathered silty clay, with roots	1B		
		1C		
	FINE TO MEDIUM SAND some silt to silty wet below 0.8 m	2	DO	2/61
183.0 1.5	Grey, hard **SILTY CLAY** a trace of sand transitioning to shale bedrock	3	DO	80
182.2 2.3	Grey, weathered **SHALE BEDROCK**	4	DO	50/5
		5	DO	50/3
180.2 4.3	**END OF BOREHOLE** **Refusal on Augers**			

Installed 50-mm Ø PVC monitoring well to 4.3 m, completed with 1.5 m screen
Sand backfill from 2.1 to 4.3 m
Bentonite seal from 0.0 to 2.1 m
Provided with a steel monument casing



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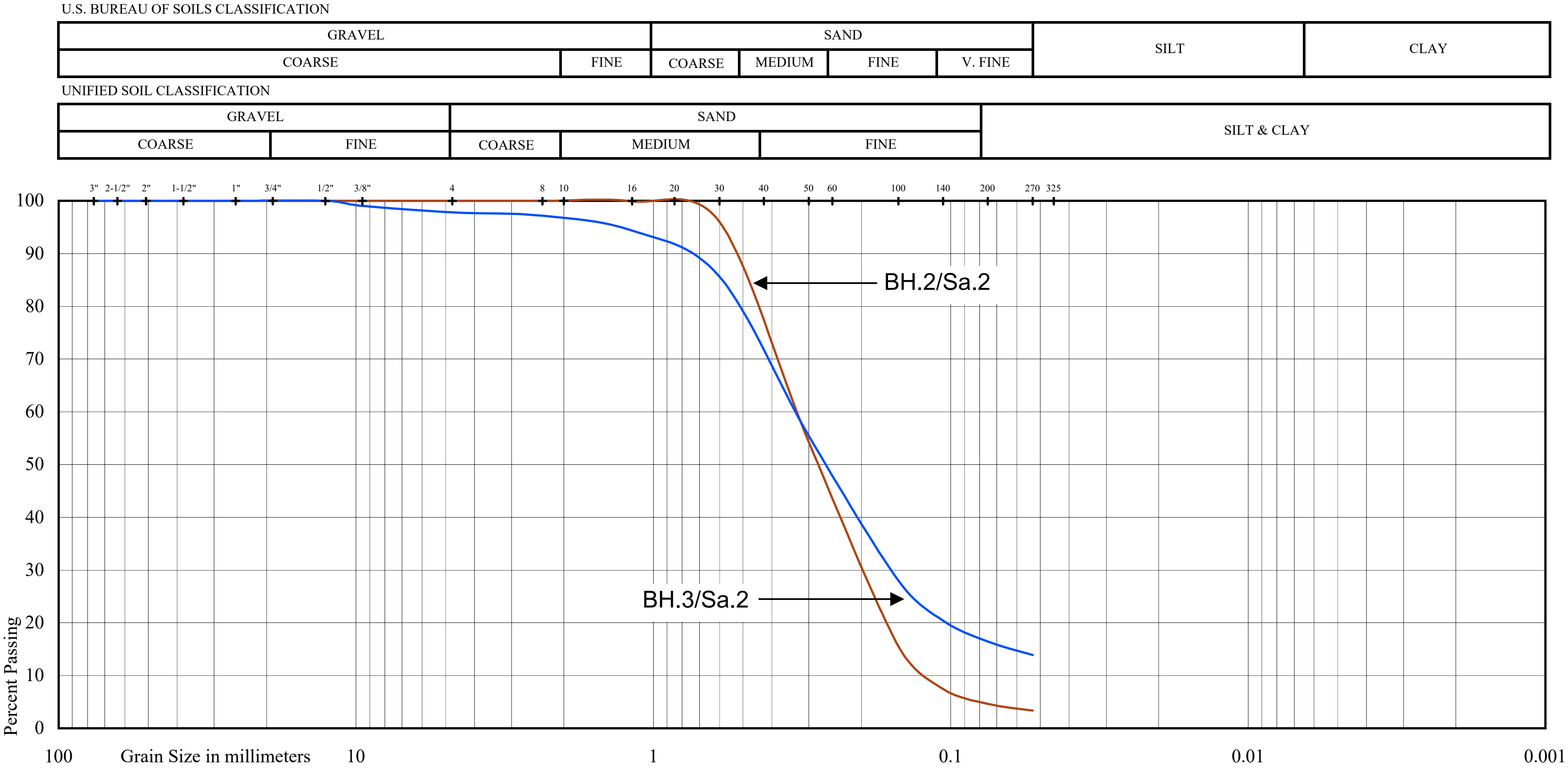
GRAIN SIZE DISTRIBUTION

Reference No: 2303-S115



Project: Proposed Residential Development

Location: Part of Lot 25, Conc. 4, Between Highway 26 and Hidden Lake, Rd., East of Barclay Blvd., Town of The Blue Mountains

BH./Sa.	2/2	3/2
Liquid Limit (%) =	-	-
Plastic Limit (%) =	-	-
Plasticity Index (%) =	-	-
Moisture Content (%) =	20	25
Estimated Permeability (cm./sec.) =	10^{-3}	10^{-3}

Borehole No:	2	3
Sample No:	2	2
Depth (m):	1	1.1
Elevation (m):	181.4	181.9

Classification of Sample [& Group Symbol]: FINE TO MEDIUM SAND

BH.2/Sa.2 - a trace of silt BH.3/Sa.2 - some silt, traces of coarse sand and gravel

Figure: 5

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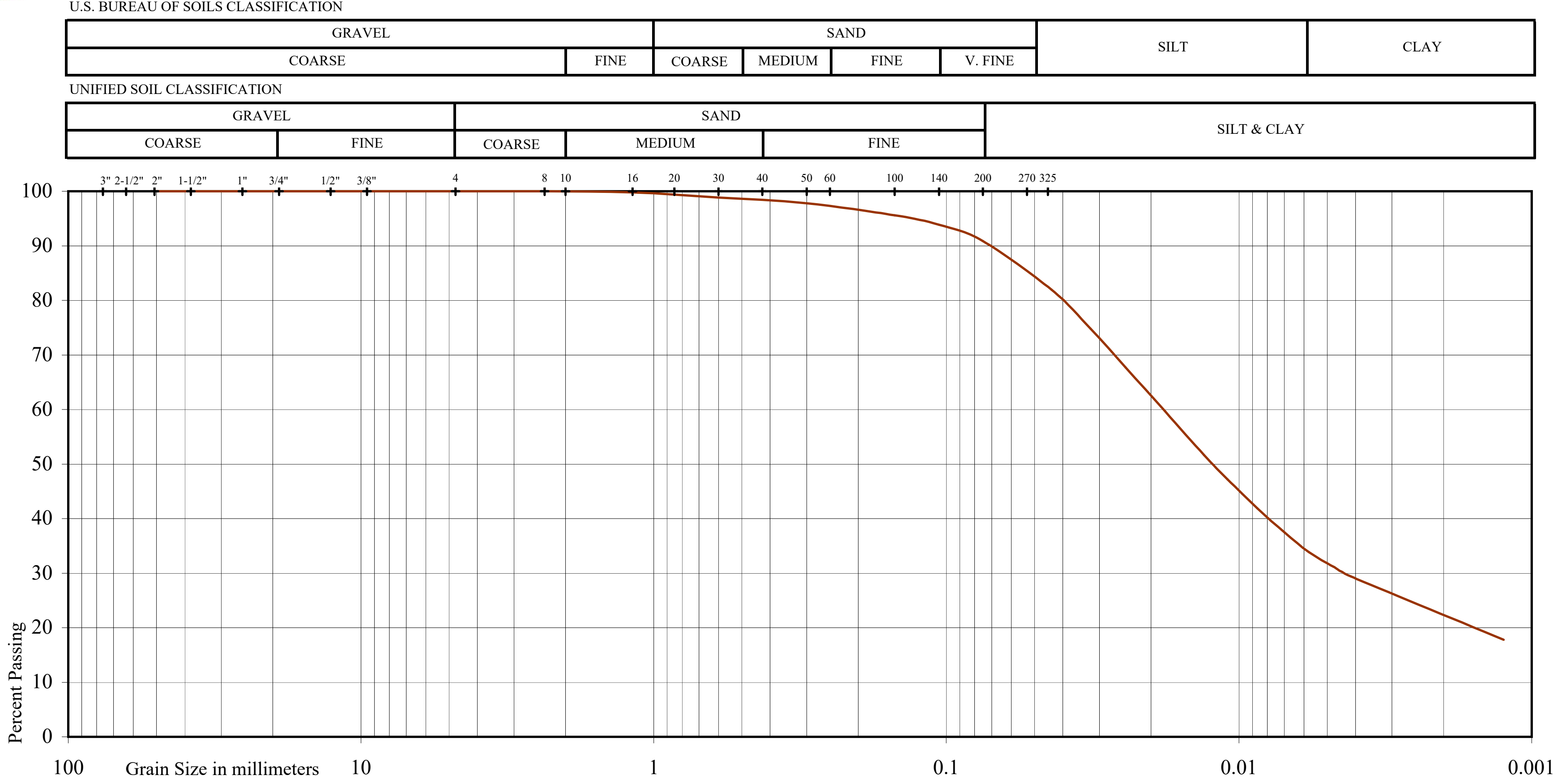
GRAIN SIZE DISTRIBUTION

Reference No: 2303-S115



Project: Proposed Residential Development

Location: Part of Lot 25, Conc. 4, Between Highway 26 and Hidden Lake, Rd., East of Barclay Blvd., Town of The Blue Mountains

Borehole No: 4

Sample No: 3

Depth (m): 1.7

Elevation (m): 182.8

Liquid Limit (%) = 28

Plastic Limit (%) = 17

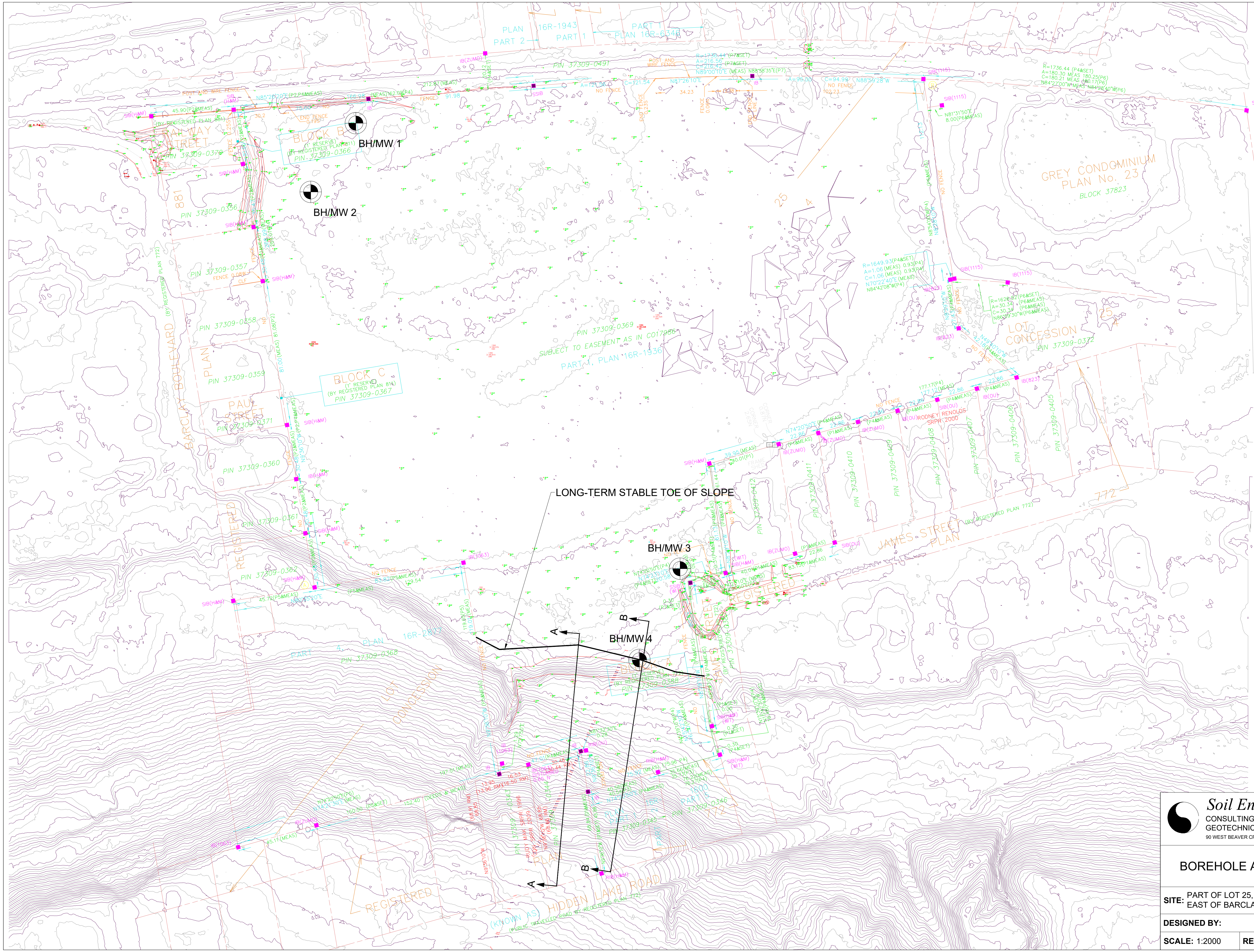
Plasticity Index (%) = 11

Moisture Content (%) = 14

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

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

Figure: 6



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

BOREHOLE AND CROSS-SECTION LOCATION PLAN

SITE: PART OF LOT 25, CONCESSION 4, B/W HIGHWAY 26 AND HIDDEN LAKE ROAD
EAST OF BARCLAY BOULEVARD, TOWN OF THE BLUE MOUNTAINS

DESIGNED BY:	**CHECKED BY:**	**DWG NO.:** 1	
SCALE: 1:2000	**REF. NO.:** 2303-S115	**DATE:** JUNE 2023	**REV** -

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SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

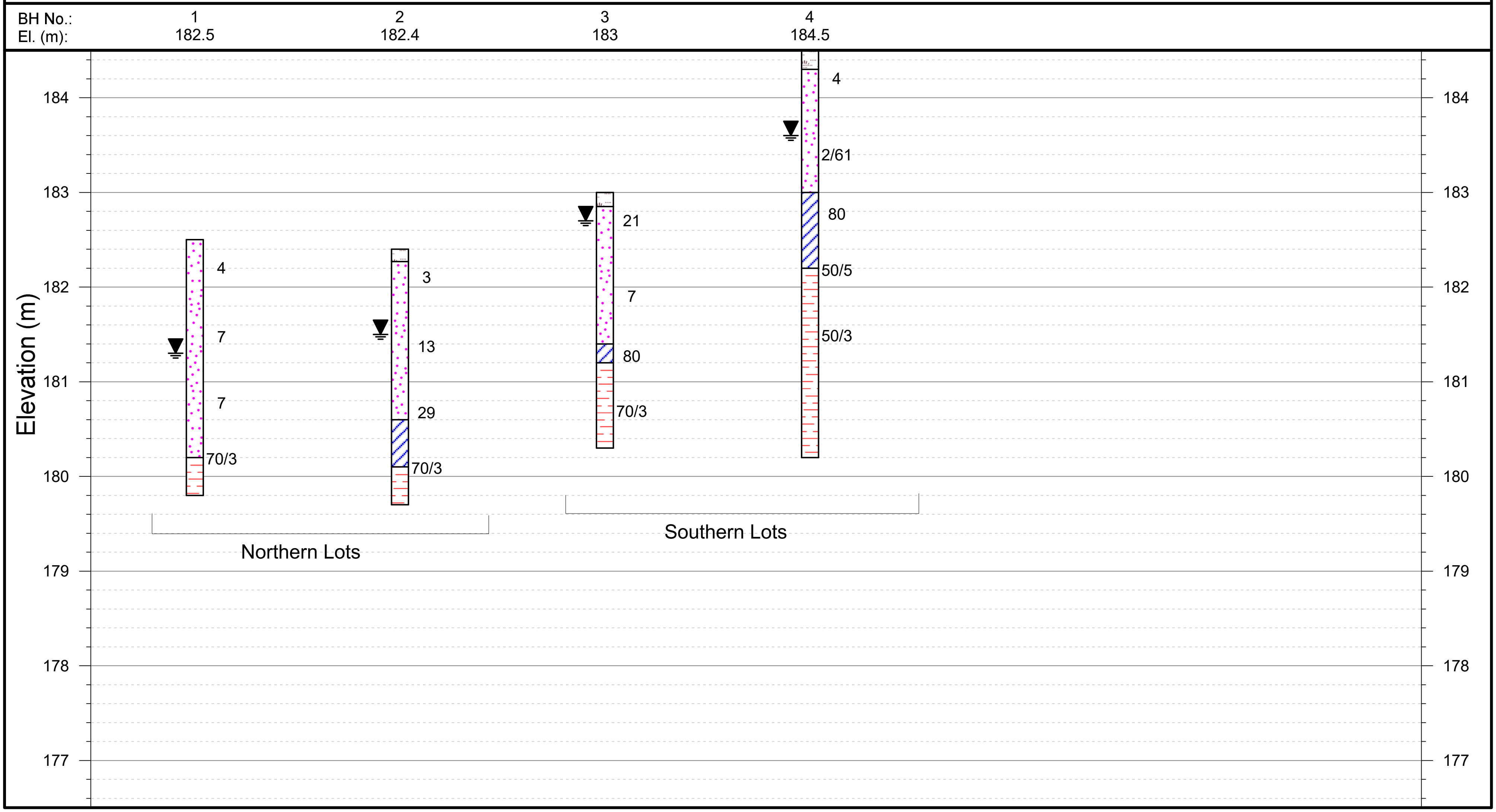
JOB NO.: 2303-S115
REPORT DATE: June 2023
PROJECT DESCRIPTION: Proposed Residential Development

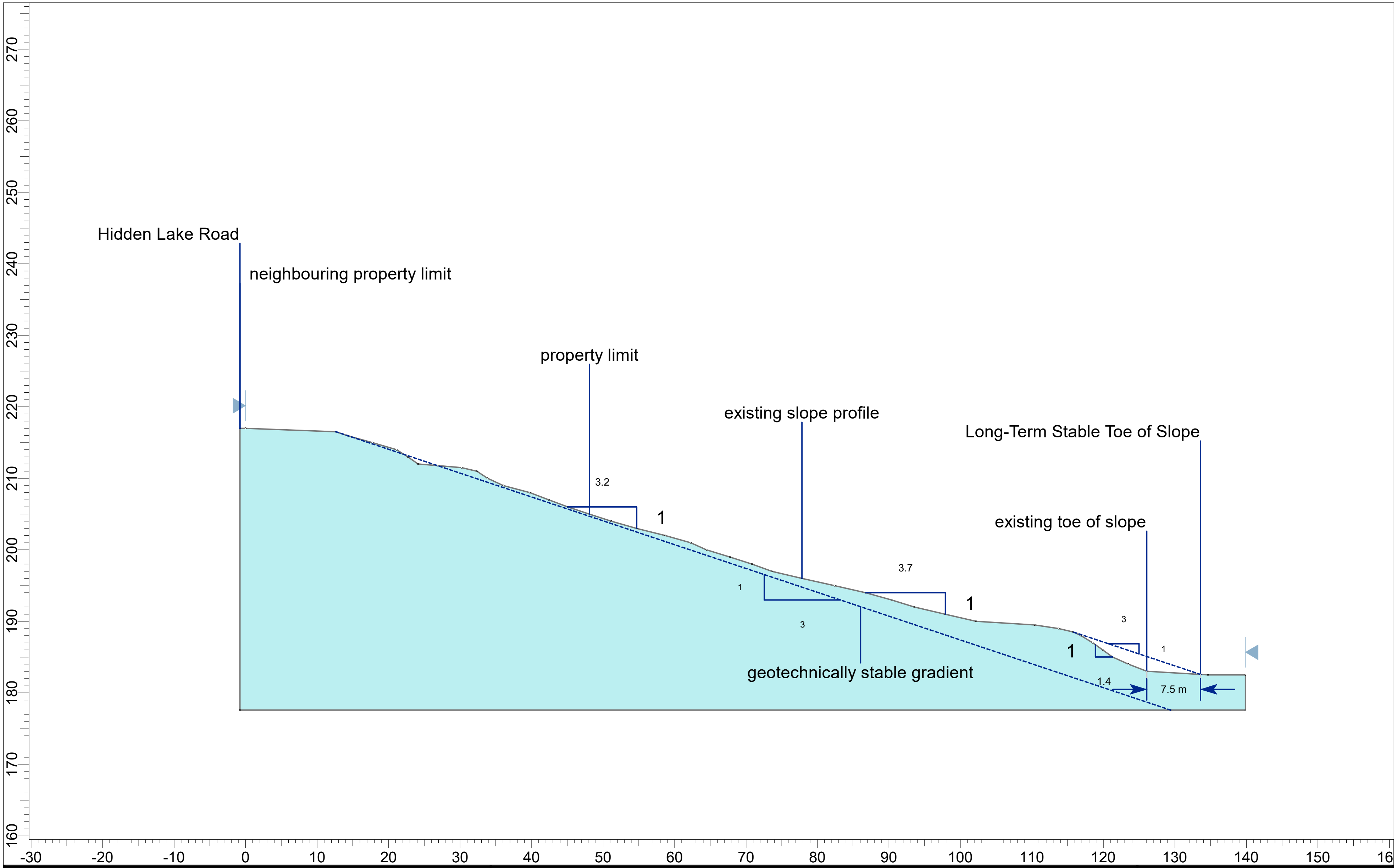
PROJECT LOCATION: Part of Lot 25, Conc. 4, B/W Hwy 26 & Hidden Lake Rd.,
East of Barclay Blvd., Town of The Blue Mountains

LEGEND

⁘ SAND ▤ SHALE ▨ SILTY CLAY ▦ TOPSOIL

▼ WATER LEVEL (STABILIZED)





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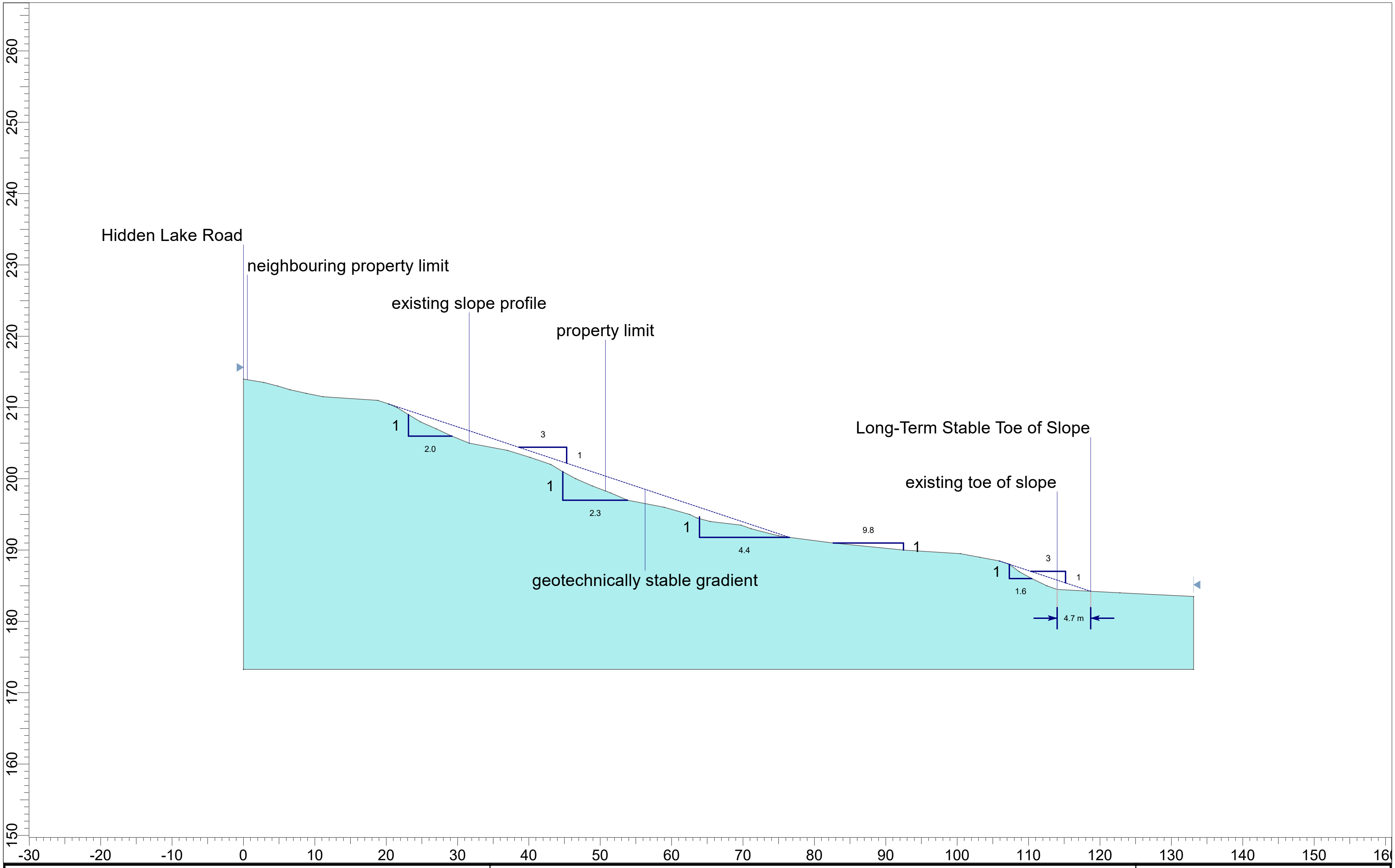
Project Title	Cross-Section A-A - Existing Slope Profile	Load Case	-
Location	Part of Lot 25, Conc. 4 - B/w Hwy 26 & Hidden Lake Rd., E of Barclay Blvd., Town of The Blue Mountains		
Drawn By HWY	Checked By KL	Scale 1:750	Revision -
Date	JUNE 2023	Reference No. 2303-S115	Drawing No. 3



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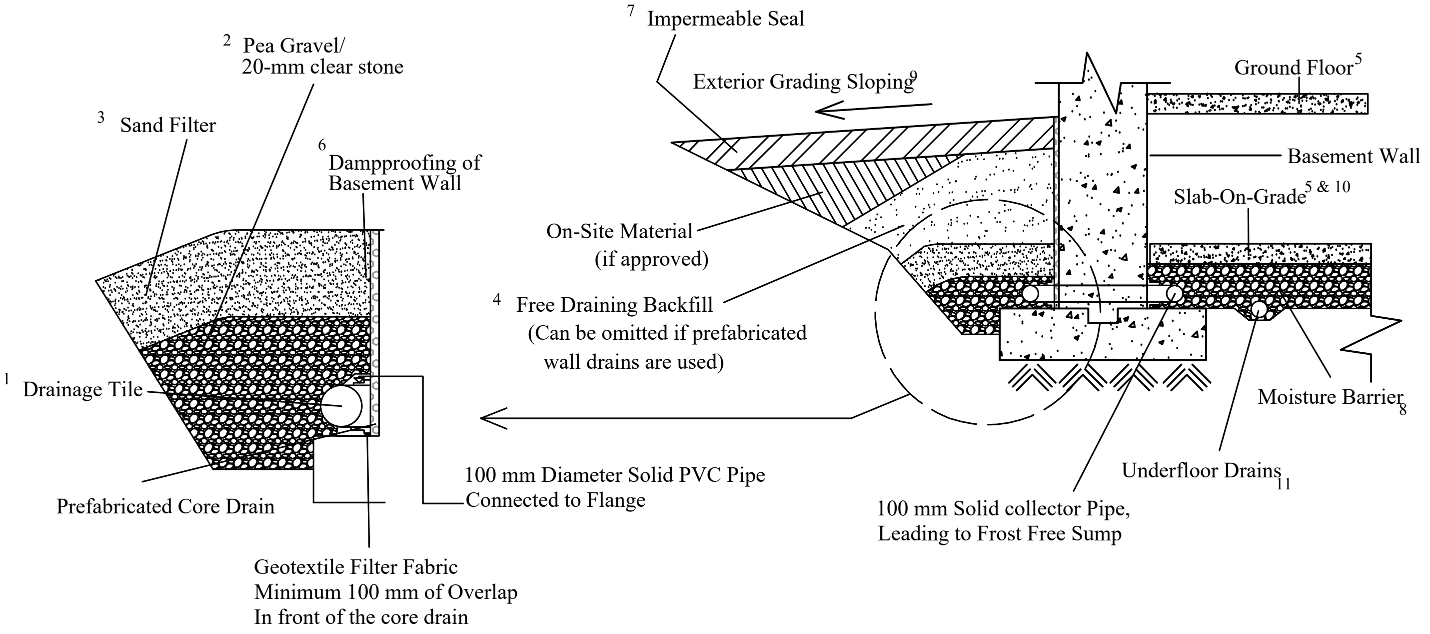
Project Title	Cross-Section B-B - Existing Slope Profile					Load Case	-
Location	Part of Lot 25, Conc. 4 - B/w Hwy 26 & Hidden Lake Rd., E of Barclay Blvd., Town of The Blue Mountains						
Drawn By	HWY	Checked By	KL	Scale	1:750	Revision	-
Date	JUNE 2023			Reference No.	2303-S115	Drawing No.	4



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 19-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 adequate 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 CRL 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 spacing should be at least 300 mm (12") between the underside of the floor slab and the top of the pipe. 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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PERMANENT PERIMETER DRAINAGE SYSTEM
(FOR OPEN EXCAVATION)

SITE: PART OF LOT 25, CONC. 4 - B/W HWY 26 & HIDDEN LAKE ROAD
EAST OF BARCLAY BLVD., TOWN OF THE BLUE MOUNTAINS

DESIGNED BY: K.L. **CHECKED BY:** B.S. **DWG NO.:** 5

SCALE: N.T.S. **REF. NO.:** 2303-S115 **DATE:** JUNE 2023 **REV** -