

Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION AND ENGINEERING DESIGN REPORT 2570-2590 ARGYLE ROAD MISSISSAUGA, ONTARIO

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1.0 THE PROJECT

Terraprobe Inc. was retained by Ranee Management (“The Client”) to conduct a subsurface investigation and provide geotechnical engineering design recommendations for the proposed redevelopment of the subject site located at 2570-2590 Argyle Road in Mississauga, Ontario. For the purposes of the description in this report, Argyle Road is considered to align in the north-south direction. The project site is located south of Dundas Street West, on the west side of Argyle Road, in Mississauga, Ontario. A site location plan is provided as Figure 1.

The site is currently occupied by two (2) high rise buildings and at-grade parking lots. The proposed developments include constructing a new high rise structure at the existing at-grade parking lot located in the western portion of the project site. The proposed building footprint will cover an area of approximately 3,600 m². The proposed structure will be fifteen (15)-storey tower, plus mechanical penthouse (stepping down to four-storey), and rest on a one-level underground parking structure (P1) with a P1 lowest finished floor elevation of Elev. 108.3 ±m.

The western portion of the project site (the existing parking lot) consists of the tableland abutting Mary Fix Creek, a tributary of Credit River. The valley slope within the study area is about 2.0 to 3.5 m high with inclination of about 1.5 to 2.0 horizontal to 1.0 vertical. A section of gabion retaining walls located along the southern portion of the slope provides the grade separation and erosion protection measure. The gabion wall is approximately 40 m in length and approximately 2.5 m in height at the southern property boundary and tapers off towards the north. The slope is generally vegetated with trees, saplings, shrub and weeds. The growth of tree trunk is generally straight and upright.

This report encompasses a geotechnical investigation of the subject site to determine the subsurface soil and ground water conditions, provide design recommendations pertaining to the geotechnical aspects of the project such as foundation design recommendations, seismic site classification, basement floor design, earth pressure design, basement drainage, excavation, shoring design, short term dewatering and other constructability recommendations.

Additionally, a detailed slope visual inspection and stability analysis is also provided in this report. Based on these studies, this report provides geotechnical engineering recommendations for the long-term stable slope crest location to help facilitate the establishment of the development limit in the area adjacent to the valley slope.

The following drawing set was provided to Terraprobe and reviewed in preparation of this engineering report:

- Plan Survey with Topography, Ref. No. 190-0076, dated May 21, 2019, by Speight, Van Nostrand & Gibson Ltd.

- Preliminary Concept Site Plan and Overall Project Statistics, Project No. 120325, dated December 24, 2019, by IBI Group.

Terraprobe is also providing a Hydrogeological Study under a separate cover (File No. 1-19-0719-46).

2.0 INVESTIGATION PROCEDURES

The field investigation was conducted on December 2 to 4, 2019. A total of ten (10) boreholes were advanced at the site by Terraprobe, and summarized below:

- Eight (8) boreholes (Boreholes 1 through 8) advanced within the approximate footprint of the proposed building;
- Two (2) boreholes (Boreholes 9 and 10) were advanced near the slope crest in order to delineate the subsurface conditions of the slope; and,
- A total of four (4) 50 mm diameter ground water monitoring wells were installed in Boreholes 2, 3, 5, and 9.

Detailed Borehole Logs are provided in Appendix A.

The boreholes were staked out in the field by Terraprobe, and were surveyed by Terraprobe for horizontal coordinates (UTM Zone 17T) and geodetic elevations (NAD 83) using a Trimble R10 Receiver connected to the Global Navigation Satellite System and the Can-Net Virtual Reference Station Network. These positions and elevations are provided on the borehole logs for the purpose of relating borehole stratigraphy and should not be used or relied on for other purposes.

The drilling work for the investigation was carried out by Profile Drilling Ltd. and was observed and recorded by Terraprobe on a full-time basis. The boreholes were advanced using a continuous flight power auger machine using solid stem augers. The Terraprobe engineer logged the boreholes and examined the samples as they were obtained. The samples obtained were sealed in clean, air-tight containers and transferred to the Terraprobe laboratory, where they were reviewed for consistency of description by a geotechnical engineer. In-house laboratory testing consisted of:

- Natural water content determination (ASTM D2216); and
- Particle size distribution (ASTM D422 and D1140).

The geotechnical laboratory results are shown on the borehole logs at the respective sampling depths, and details of the results are provided in Appendix B.

The samples were obtained using the Split-Barrel Method (ASTM D1586). The samples were taken at intervals and there is consequently some interpolation of the borehole layering between samples. The boundaries between the various strata represent an inferred transition rather than a precise plane of geological change. The subsurface conditions have been confirmed in a series of widely spaced boreholes, and will vary between and beyond the borehole locations.

The water levels were monitored in the open boreholes upon completion of drilling, as noted on the enclosed borehole logs. The PVC well is slotted near the base and fitted with a bentonite clay seal as shown on the accompanying borehole logs.

3.0 SUBSURFACE CONDITIONS

The subsurface soil and ground water conditions encountered in the boreholes are presented on the attached Borehole Logs in Appendix A. The stratigraphic boundaries indicated on the geotechnical Borehole Logs are inferred from non-continuous samples and observations of drilling resistance and typically represent a transition from one soil type to another. These boundaries should not be interpreted to represent exact planes of geological change. The subsurface conditions have been confirmed in a series of widely spaced boreholes, and will vary between and beyond the borehole locations. The discussion has been simplified in terms of the major soil strata for the purposes of geotechnical design.

3.1 Soil Stratigraphy

The following stratigraphy is based on the borehole findings, as well as the geotechnical laboratory testing conducted on selected representative soil samples.

3.1.1 Pavement Structure and Earth Fill

Asphaltic concrete was encountered from the surface at all borehole locations. The thickness of the asphaltic concrete ranges from 75 to 90 mm. An aggregate base course with a thickness of 50 mm was encountered in Borehole 3 underlying the asphaltic concrete.

Underlying the pavement structure, the boreholes encountered a layer of earth fill extending to depths ranging between 0.8 and 2.3 m below grade (Elev. 110.0 to 111.4 ±m). The earth fill is variable in composition across the site, but is predominantly clayey silt, trace sand, and trace gravel. Silty sand, trace clay trace gravel was encountered in Borehole 6. Shale fragments, organics, asphaltic concrete and brick debris are also encountered at various borehole locations. A strong hydrocarbon odour was noted in Borehole 1 at a depth of 1.5 m below surface grade. The earth fill ranges in colour from dark grey to brown, and is generally moist. Due to the variation and inconsistent placement of the earth fill materials, the relative density of the earth fill varies from loose to compact. The moisture contents of the earth fill samples range from 9 to 27% by mass, indicating a moist to wet condition.

3.1.2 Clayey Silty Till

Underlying the earth fill, the boreholes encountered a cohesive deposit of clayey silt till. The clayey silt was encountered at depths ranging from 0.8 to 2.3 m below grade (Elev. 110.0 to 111.4 ±m), and extends to depths ranging from 1.5 to 3.5 m below surface grade (Elev. 108.6 to 110.4 ±m).

Glacial till is typically a heterogeneous mixture of all grain sizes. At this site the till is composed predominantly of grey or greyish brown clayey silt, sandy or some sand, and trace gravel.

SPT N-Values recorded in the clayey silt till range from 7 to over 50 blows per 300 mm of penetration, indicative of a firm to hard consistency. The moisture contents of the clayey silt till deposit samples range from 8 to 19% by mass, indicating a moist condition.

Three (3) grain size distribution tests were conducted on select samples of the clayey silt till. The results of the testing are provided in Appendix B.

3.1.3 Inferred Bedrock

Split spoon refusal was encountered at all borehole locations underlying the clayey silt glacial till. Based on drilling observations and the grey shale fragments within the split spoons, the refusal is likely encountered on inferred bedrock of the Georgian Bay Formation. Augering was advanced passed the initial split spoon refusal in Boreholes 2, 3, 5, and 9 in order to install and seal ground water monitoring wells at a deeper elevations. All boreholes terminated within the inferred bedrock. The depths of the top of inferred bedrock are summarized in the following table.

Borehole No.	Depth to Inferred Bedrock (m)	Elevation (m)
1	3.0	110.4
2	3.5	109.8
3	3.5	109.6
4	2.3	110.2
5	3.0	109.6
6	2.3	109.8
7	3.0	108.6
8	2.3	109.2
9	1.5	110.2

Borehole No.	Depth to Inferred Bedrock (m)	Elevation (m)
10	3.0	109.9

Bedrock of the Georgian Bay Formation is a deposit predominantly comprised of laminated to thinly bedded grey shale of Ordovician age. The formation contains interbeds of light grey calcareous shale, limestone/dolostone, and calcareous sandstone which are discontinuous and nominally 50 to 300 mm thick. Shale is a relatively low strength rock type, whereas the limestone/dolostone beds are considered medium strength rock.

A summary of properties with respect to the shale within the Georgian Bay Formation was presented in the Ontario Ministry of Transportation and Communications document RR229, *Evaluation of Shales for Construction Projects* (March 1983), as follows:

	Uniaxial Compressive Strength (MPa)	Young's Modulus (GPa)	Dynamic Modulus (GPa)	Poisson's Ratio
Average	28	4	19	0.19
Range	8 to 41	0.5 to 12	6 to 38	0.1 to 0.25

There is typically a zone of weathering at the contact between the rock of the Georgian Bay Formation and the glacial soil overburden. In the Ontario Ministry of Transportation and Communications document RR229, *Evaluation of Shales for Construction Projects*, there is reproduced from Skempton, Davis and Chandler, a *typical weathering profile of a low durability shale*, that characterizes the shale surface into three grades of weathering and four zones described as follows:

	Zone	Description	Notes
*Fully Weathered	IVb	soil like matrix only	indistinguishable from glacial drift deposits, slightly clayey, may be fissured
Partially Weathered	IVa	soil like matrix with occasional pellets of shale less than 3 mm dia.	little or no trace of rock structure, although matrix may contain relic fissures
	III	soil like matrix with frequent angular shale particles up to 25 mm dia.	moisture content of matrix greater than the shale particles
	II	angular blocks of unweathered shale with virtually no matrix separated by weaker chemically weathered but intact shale	spheroidal chemical weathering of shale pieces emanating from relic joints and fissures, and bedding planes
Unweathered (Sound)	I	shale	regular fissuring

The augered borehole method used at this site is conventionally accepted investigative practice. However, the interval sampling method does not define the bedrock surface with precision, particularly where the surface of the rock is weathered, weaker and easily penetrated by auger. The auger refusal is generally indicative of a presence of a relatively less weathered/sound shale and/or limestone/dolostone layers. It

should be noted that confirmation and characterization of the bedrock through rock coring was not included in our scope of work. Therefore, the bedrock surface elevations at the borehole locations, as noted on the borehole logs, could not be confirmed, and were inferred from the borehole augering, auger grinding, split barrel sampler refusal and bouncing. Auger grinding or sampler refusal in this case could either be inferred as bedrock or could be due to the presence of boulders/obstruction/limestone slabs which may be present within the overburden, therefore actual bedrock surface elevations may vary from the inferred elevations noted on the borehole logs. It must be noted that inference of bedrock level based on auger grinding and/or sampler refusal does not provide bedrock level accurately. Any variation in the design bedrock level and actual bedrock level may result in significant cost implications and schedule delays (including redesign and additional construction costs) for the project.

3.2 Ground Water

Unstabilized ground water level observations were made in the open boreholes during and after drilling, as noted on the borehole logs. A total of four (4) monitoring wells was installed in select boreholes to facilitate long-term ground water monitoring. A summary of the ground water observations is provided in the table below along with the most recent ground water measurement.

Borehole	Depth / Elevation (m)	Depth to (m)		Strata Screened	Water Depth / Elevation (m)		
		Cave	Unstabilized water level		Highest Level (m)	Date	Level Range (m)
1	3.2 / 110.2	open	dry	Well not installed			
2	6.4 / 106.9	open	dry	Inferred Bedrock	4.9 / 108.4	Jan 9, 2020	4.6 to 4.9 108.4 to 108.7
3	4.9 / 109.6	open	dry	Inferred Bedrock	2.8 / 110.3	Jan 9, 2020	2.5 to 2.8 110.3 to 110.6
4	3.1 / 110.2	open	dry	Well not installed			
5	6.1 / 109.6	open	dry	Inferred Bedrock	2.8 / 109.8	Jan 9, 2020	2.5 to 2.8 109.8 to 110.2
6	2.4 / 109.8	open	dry	Well not installed			
7	3.1 / 108.6	open	dry	Well not installed			
8	2.4 / 109.2	open	dry	Well not installed			
9	3.7 / 108.0	open	dry	Inferred Bedrock	2.7 / 109.0	Jan 9, 2020	1.7 to 2.7 109.0 to 110.0
10	3.1 / 109.9	open	dry	Well not installed			

Based upon the observations in the boreholes and monitoring wells, the ground water table is approximately at Elev. 110 ±m in the in the tableland. It should be noted that regrading of the site, construction dewatering,

building drains or dewatering systems, seasonal fluctuations and creek surface water may cause significant changes to the depth of the ground water table over time.

Additional information pertaining to ground water at the site is discussed in the Hydrogeological Study by Terraprobe under a separate cover (File No. 1-19-0719-46).

4.0 GEOTECHNICAL ENGINEERING DESIGN (BUILDING)

The following discussion and recommendations are based on the factual data obtained from this investigation, and are intended for use of the owner and the design engineer. Contractors bidding or providing services on this project should review the factual data and determine their own conclusions regarding construction methods and scheduling.

This report is provided on the basis of these terms of reference and on the assumption that the design features relevant to the geotechnical analyses will be in accordance with applicable codes, standards and guidelines of practice. If there are any changes to the site development features or any additional information relevant to the interpretations made of the subsurface information with respect to the geotechnical analyses or other recommendations, then Terraprobe should be retained to review the implications of these changes with respect to the contents of this report.

4.1 Foundation Design Parameters

The proposed development consists of constructing a residential development of a 15-storey building with a podium structure of 9 storeys, with 4 floors of above grade parking, and 1 level of underground parking (P1). The structure will have an estimated lowest FFE of Elev. 108.3 ±m, approximately 3.8 m below established grade (112.1 ±m), within the inferred bedrock. At this elevation, conventional shallow foundations would be made to bear on bedrock at Elev. 108 ±m.

Bedrock was not cored and proven at this site, and inferred based on drilling observations. Therefore, the depth to sound bedrock was not determined by our investigation. It is therefore required that Terraprobe inspect and approve the foundation subgrade as sound bedrock in order for the following recommendations to be valid.

Spread footings on sound Georgian Bay Formation bedrock may be designed using a maximum factored geotechnical resistance at ultimate limit state (ULS) of 10,000 kPa.

The SLS bearing is a function of acceptable total and differential settlement. The net geotechnical reaction at SLS should be limited to 6,000 kPa for spread footings. The settlement of foundations made on the sound bedrock is elastic, linear and non-recoverable. The settlement occurs as load is applied. Load tests carried out in the Georgian Bay Formation have indicated that the rock formation has predictable and similar

response to loading over its area of occurrence. These tests have yielded parameters to estimate the elastic compression of the rock under applied loading. This compression is a function of the pressure applied and the size of the area loaded. To estimate the settlement of foundations of different sizes and assess differential settlement between foundation units, the following relationship can be used:

$$\delta = 1000q_{sLS} \left[\frac{2}{1 + 0.4/B_f} \right]^2 \frac{1}{k}$$

Where:

δ	=	estimated vertical displacement in the rock beneath the centre of the loaded foundation (mm)
q_{sLS}	=	applied bearing pressure on the rock at the base of the foundation (kPa)
B_f	=	the nominal foundation width (m)
k	=	modulus of displacement (kPa/m): 600,000 kPa/m for sound bedrock

Footings stepped from one level to another must be at a slope not exceeding 7 vertical to 10 horizontal. The design earth cover for frost protection of foundations exposed to ambient environmental temperatures is 1.2 meters in the Greater Toronto area. At locations adjacent to ventilation shafts, it is normal practice to provide insulation to ensure that foundations are not affected by the cold air flow.

Prior to pouring concrete for the footings, the footing subgrade must be cleaned of all deleterious materials such as softened, disturbed, or caved material, as well as any weathered bedrock or standing water. Zones of rubblized bedrock encountered at the footing elevation must be sub-excavated, and foundations must be made to bear on sound bedrock. Terraprobe must inspect and approve the proposed footing subgrades to confirm sound bedrock is encountered at the founding elevations. If construction proceeds during freezing weather conditions, temporary frost protection for the footing bases and concrete must be provided. It should be noted that the bedrock surface can weather and deteriorate on exposure to the atmosphere or surface water. Therefore, foundation bases that will remain open and exposed for an extended period of time should be protected by applying a skim coat of lean concrete.

4.2 Earthquake Design Parameters

The Ontario Building Code (2012) stipulates the methodology for earthquake design analysis, as set out in Subsection 4.1.8.7. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration and the site classification.

The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4A of the Ontario Building Code (2012). The classification is based on the determination of the average shear wave velocity in the top 30 metres of the site stratigraphy, where shear wave velocity (v_s) measurements have been taken. Alternatively, the classification is estimated on the basis of rational analysis of undrained shear strength (s_u) or penetration resistance (N-values).

$$v_{s-avg} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{v_{si}}}$$

Shear wave velocity

$$s_{u-avg} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{s_{ui}}}$$

Undrained shear strength

$$N_{avg} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}}$$

SPT N-values

For spread footings resting uniformly on sound (unweathered) bedrock, it is recommended that the site designation for seismic analysis is Class B, as per Table 4.1.8.4.A of the Ontario Building Code (2012). Tables 4.1.8.4.B and 4.1.8.4.C. of the same code provide the applicable acceleration and velocity based site coefficients.

Site Class	Values of F_a				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) \geq 1.25$
B	0.8	0.8	0.9	1.0	1.0

Site Class	Values of F_v				
	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \geq 0.5$
B	0.6	0.7	0.7	0.8	0.8

4.3 Earth/Rock Pressure Design Parameters

4.3.1 Earth Pressure Design Parameters

The appropriate values for use in the design of structures subject to unbalanced earth pressures at this site are tabulated as follows:

Stratum/Parameter	ϕ	γ	K_a	K_o	K_p
Compact Granular Fill, Granular 'B' (OPSS 1010)	32	21.0	0.31	0.47	3.25
Existing Earth Fill	28	19.0	0.36	0.53	2.77
Native Clayey Silt Till	30	21.0	0.33	0.50	3.00
Georgian Bay Formation (Bedrock)	28	26.0	n/a	n/a	n/a

Where: γ = bulk unit weight of soil (kN/m³)

φ	=	internal angle of friction (degrees)
K_a	=	Rankine active earth pressure coefficient (dimensionless)
K_o	=	Rankine at-rest earth pressure coefficient (dimensionless)
K_p	=	Rankine passive earth pressure coefficient (dimensionless)

The above earth pressure parameters pertain to a horizontal grade condition behind a retaining structure. Values of earth pressure parameters for an inclined retained grade condition will vary.

Walls subject to unbalanced earth pressures must be designed to resist a pressure that can be calculated based on the following equation:

$$P = K[\gamma(h - h_w) + \gamma' h_w + q] + \gamma_w h_w$$

Where,	P	=	the horizontal pressure at depth, h (m)
	K	=	the earth pressure coefficient
	h_w	=	the depth below the ground water level (m)
	γ	=	the bulk unit weight of soil, (kN/m ³)
	γ'	=	the submerged unit weight of the exterior soil, ($\gamma - 9.8$ kN/m ³)
	q	=	the complete surcharge loading (kPa)

The wall backfill must be drained effectively to eliminate hydrostatic pressures on the wall that would otherwise act in conjunction with the earth pressure. In this case, the above equation is simplified to:

$$P = K[\gamma h + q]$$

To ensure that there is no hydrostatic pressure acting in conjunction with the earth pressure, where the structure is made directly against a shored excavation, drainage is provided by forming a drained cavity with prefabricated drain core material covering the excavation face and designed to discharge collected water into a perimeter/underfloor drainage system.

Consideration must also be given to the possible effects of frost on structures retaining earth. Pressures induced by freezing in frost-susceptible soils exert pressures and are effectively irresistible.

4.3.2 Rock Pressure

The empirical approach for the design of foundation walls below bedrock level has been to use a uniform pressure distribution for the design of the basement walls below the top of bedrock elevation, which is consistent with the maximum earth pressure calculated for the lowest level of soil in the profile. This approach is likely conservative but it recognizes the practical requirement to have a foundation wall of a consistent width through the lower reach of the building.

If excavation proceeds into the sound bedrock, this approach does not recognize the potential for pressures on the basement wall due to time dependant rock swell that results when locked in horizontal stresses are released. This is typically not required within the upper 2 m of the bedrock due to weathering in the Georgian Bay Formation.

Therefore, if excavation proceeds into sound bedrock, sufficient time between cutting of the rock face and construction of the building structure to allow the rock to de-stress and swell should be allowed. Experience suggests that if there is a 120-day period after the rock cut, before the rock is restrained by the structure, that there has been sufficient swell and no significant stresses are imposed on the structural wall. Depending on the building construction sequence some provision for compressible material at the foundation perimeter in rock may be necessary.

Where pits are made for sumps and elevators or other such features which are incorporated within the major excavation, there must be careful consideration of the potential for rock squeeze effects if the pits are to be cast directly against the rock face, if excavation proceeds into sound bedrock. For such structures, a crushable layer can be placed between the rock and the concrete. Ethafoam is typically used in this application and the walls are designed for 25% to 50% compressive strength of the foam. At 50% compression 220 Ethafoam plank material will provide a resistance of 18 psi (124 kPa). At 25% compression 220 Ethafoam plank material will provide a resistance of 9 psi (65 kPa). The 10% compression of this material is 7 psi (50 kPa), which will allow for concrete placement. Alternatively, if the rock is over excavated by at least 600 mm and the pits and sumps are backfilled with 19 mm clear stone (OPSS.MUNI 1004), then there is sufficient give in the clear stone backfill to accommodate the rock swell.

Rock squeeze effects are not relevant to spread footing foundation excavations as the foundation concrete strength exceeds any rock squeeze pressures.

4.3.3 Sliding Resistance

The geotechnical resistance to sliding of earth retaining structures is developed by friction between the base of the footing and the soil or rock. This friction (**R**) depends on the normal load on the rock contact (**N**) and the frictional resistance of the rock (**tan φ**) expressed as: **R = N tan φ**. This is an unfactored resistance. The factored resistance at ULS is **R_f = 0.8 N tan φ**.

4.4 Basement Floor Slab Design Parameters

The P1 level slab is to be made near Elev. 108.3 ±m on inferred bedrock of the Georgian Bay Formation, which is suitable for the support of a slab. The modulus of subgrade reaction appropriate for design of the slab resting on sound bedrock is 80,000 kPa/m.

It is necessary that building floor slabs be provided with a capillary moisture barrier and drainage layer. This is made by placing the slab on a minimum 300 mm layer of 19 mm clear stone (OPSS.MUNI 1004)

compacted by vibration to a dense state. Basement drainage is required as discussed in the following Section 4.5.

4.5 Basement Drainage

A separate hydrogeological report will be prepared by Terraprobe for this site (File. No. 1-19-0719-46), which provides the approximate amount of daily permanent ground water collection and discharge.

To assist in maintaining dry basements and preventing seepage, it is recommended that exterior grades around the building be sloped away at a 2 percent gradient or more, for a distance of at least 1.2 metres. Provision of nominal subfloor drainage is required in conjunction with the perimeter drainage of the structure, to collect and remove the water that infiltrates at the building perimeter and under the floor. Perimeter and subfloor drainage are required throughout below grade areas (Figures 3A, 3B and 4).

It is recommended that the subfloor drainage system consists of minimum 100 mm diameter perforated pipes spaced at a maximum of 6 metres on centre. The pipes must be surrounded on all sides by a minimum of 100 mm of 19 mm clear stone, and the pipe inverts should be a minimum 300 mm below the base of the slab. The elevator pits can be drained separately with an independent lower pumping sump or can be designed as water proof structures which are below the drainage level. It is recommended to cut the rock subgrade neat to 300 mm beneath the floor slab and place subdrains directly on the subgrade. The subfloor drainage layer is then comprised of 300 mm of 19 mm clear stone (OPSS.MUNI 1004).

Prefabricated drainage composites, such as Miradrain 6000 (Mirafi) or Terradrain 200 (Terrafix), should be incorporated between the shoring wall or rock face and the cast-in-place concrete foundation wall to make a drained cavity. Drainage from the cavity must be collected at the base of the wall in non-perforated pipes and conveyed directly to the sumps. The flow to the building sump from the subsurface drainage will be governed largely by the building perimeter drainage collection during rainfall and runoff events. Typical shored excavation drainage details are provided in Figure 3B. A compressible layer may also be required, depending if the bulk excavation extends into sound bedrock, to accommodate rock squeeze.

The drainage system is a critical structural element, since it keeps water pressure from acting on the basement floor slab or on the foundation walls. As such, the sump that ensures the performance of this system must have a duplexed pump arrangement for 100% pumping redundancy and these pumps must be on emergency power. The size of the pump should be adequate to accommodate the anticipated ground water and storm event flows. It is expected that the seepage can be controlled with typical widely available commercial sump pumps.

4.6 Excavations

Excavations must be carried out in accordance with the Occupational Health and Safety Act, Ontario Regulation 213/91 (as amended), Construction Projects, Part III – Excavations, Sections 222 through 242. These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety. For practical purposes, the earth fill and all overburden clayey silt till should be considered Type 3 soils.

Where workers must enter excavations advanced deeper than 1.2 m, the trench walls should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. The regulation stipulates maximum slopes of excavation by soil type as follows:

Soil Type	Base of Slope	Steepest Slope Inclination
1	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
2	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
3	from bottom of trench	1 horizontal to 1 vertical
4	from bottom of trench	3 horizontal to 1 vertical

Minimum support system requirements for steeper excavations are stipulated in the Occupational Health and Safety Act and Regulations for Construction Projects, and include provisions for timbering, shoring and moveable trench boxes.

Excavations made in bedrock can be vertical, provided the rock faces are scaled and maintained to preclude the possibility of spalls. Where this is not possible, protective mesh can be draped over the rock face when work is required in the area immediately beside the cut rock face.

The overburden soils can be removed by conventional excavation equipment. The presence of cobbles/boulders is possible within the earth fill or native till overburden. Fragments of shale and limestone are also likely to be encountered in the transition zone between the native soil and the bedrock. The size and distribution of cobbles/boulders or bedrock fragments cannot be predicted with boreholes, as the sampler size is insufficient to secure representative particles of this size. The bedrock below the site, while likely predominantly shale, contains harder beds. It is likely that some thick layers of hard limestone/dolostone may be encountered. The risk and responsibility for the removal and disposal of cobbles/boulders/obstructions, and the removal or penetration of these harder layers must be addressed in the contract documents for foundations, excavations and shoring.

The Georgian Bay Formation rock can be removed with conventional excavation equipment once it has been displaced by a ripper tooth or a hoe ram. The hard layers of limestone/dolostone within the shale

formation are normally broken with hoe mounted hydraulic rams before excavation. Excavating detailed shapes for foundations and the edges of the excavation are normally accomplished with hoe mounted hydraulic rams.

Where a harder layer coincides with the foundation level, it may be necessary to remove the entire thickness of the hard layer to expose the founding level. It is impractical to remove a portion of one of these layers. This can result in vertical overbreak not intrinsic to the project requirements. The risk and responsibility for the potential overbreak under these circumstances and the supply and placement of the additional concrete to restore the foundation grade must be addressed in the contract documents for foundation and excavation contractors.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the exposed rock in the foundation excavations is required. The rock beneath this site is susceptible to frost damage. Consideration must be given to frost effects, such as heave or softening, on exposed surfaces in the context of this particular project development. If foundation construction proceeds during the hot summer months, extreme heat has also been found to cause relatively rapid degradation of exposed rock surfaces. Depending on the weather at the time of construction it could be necessary to make final rock cuts for foundations and immediately seal these cuts with a concrete skim coat to preserve the integrity of the bearing surface.

Georgian Bay Formation shale has been known to issue gases (methane and hydrogen sulphide) when penetrated, although this was not observed in the boreholes at this site. Nominal ventilation of underground parking structures typically addresses this issue. This is commonly handled the same way as excessive CO gas is handled in underground parking levels, using servo-controlled blowers (fans) which are automatically started by gas detectors.

4.7 Ground Water Control

Ground water control and considerations pertaining to ground water and drainage are discussed in the Terraprobe upcoming hydrogeological report for the site under a separate cover (File No. 1-19-0719-46).

For design purposes, the design ground water table is at about Elev. 110 ±m. As such, the excavation to the proposed excavation and basement level will extend below the stabilized ground water table.

In considering the approach to ground water control during construction at this site, the shoring for the excavation will consist of permeable soldier pile and lagging walls. The shoring walls should be toed into sound bedrock of the Georgian Bay Formation.

The clayey silt till and sound bedrock is considered to be of low permeability. In general, the volume of water anticipated to flow into the open excavation is such that temporary pumping from the excavation using a conventional sump pump arrangement is expected to suffice for the control of ground water.

The Ministry of the Environment and Climate Change (MOECC) has recently made changes to the requirement for Permit to Take Water approvals for construction related activities. Under the revised requirements, specific construction-related water-taking activities are eligible for Environmental Activity and Sector Registry (EASR). The trigger volume for EASR registration is a water taking of more than 50,000 litres/day. This includes the ground water that is collected in the open excavation as well as any precipitation and/or surface runoff that enters the excavation.

4.8 Shoring Design

The site is bounded on all sides by the parking lot servicing the existing residential buildings on the property. The residential properties are located north of the proposed building footprint, a distance ranging from 16.5 to 17.0 m. No excavation shall extend below the foundations of existing adjacent structures without adequate alternative support being provided. Terraprobe recommends that if the existing footings for the adjacent buildings are not on bedrock and they are within the zone of influence of the shoring system, they may be supported using a continuous interlocking caisson wall shoring or may be underpinned down to bedrock at locations adjacent to the proposed deeper excavation. Underpinning guidelines are provided as Figure 5. Where excavations cannot be sloped, they can be supported using a shoring system such as soldier piles and lagging shoring.

Exposed rock faces will weather and deteriorate. It may be necessary to provide draped steel mesh over the excavation faces to protect workmen beneath the rock faces from spalls. The mesh directs small rock spalls down the face and precludes toppling of any significant size pieces of material.

The shoring system would best be supported by pre-stressed soil anchors extending beneath the adjacent lands. Pre-stressed anchors are installed and stressed in advance of excavation and this limits movement of the shoring system as much as is practically possible. The use of anchors on adjacent properties requires the consent of the adjacent land owners, expressed in encroachment agreements. The City Transportation and Works Department negotiates “permits” for the encroachment in City lands, which are generally allowed.

If a shoring system is to be used to provide ground water control, the entire excavation could be constructed using continuous interlocking caisson wall shoring, cut off within the sound bedrock. Further recommendations for caisson shoring design requirements can be provided upon request.

4.8.1 Earth Pressure Distribution

If the shoring is supported with a single level of earth anchor or bracing, a triangular earth pressure distribution similar to that used for the basement wall design is appropriate, and is defined by:

$$P = K[\gamma H + q]$$

Where,	P	=	the horizontal pressure at depth, H (kPa)
	K	=	the earth pressure coefficient (see Section 4.3.1)
	H	=	the total depth of the excavation (m)
	γ	=	the bulk unit weight of soil, (kN/m ³)
	q	=	the complete surcharge loading (kPa)

The bedrock induces no pressure on shoring systems. The requirement for lagging support of partially weathered rock depends on the cleanliness of the excavation break.

4.8.2 Soldier Pile Toe Design

Soldier pile toes should be made in sound bedrock of the Georgian Bay Formation. The factored vertical geotechnical resistance at ULS for the design of a pile, embedded in the sound bedrock, is 10 MPa. The factored lateral geotechnical resistance at ULS of the sound rock is 1 MPa.

The exposed Georgian Bay Formation deteriorates with time. Exposed excavation faces have been found to flake and recede as much as 300 mm with 12 months exposure. This recession generally takes the form of coin-size shale particles dropping from the face on a constant basis. The deteriorated rock loses internal integrity and bearing capability. Typically, the piles advanced as part of the shoring wall are advanced at least 1 m below the base of the excavation to accommodate this weathering, to ensure the lateral and vertical capacities provided can be utilized.

4.8.3 Shoring Support

If anchor support is necessary and determined to be feasible, the shoring system should be supported by pre-stressed soil anchors extending beneath the adjacent lands. Pre-stressed anchors are installed and stressed in advance of excavation and this limits movement of the shoring system as much as is practically possible. The use of anchors on adjacent properties requires the consent of the adjacent land owners, expressed in encroachment agreements.

Conventional earth anchors could be made with a continuous hollow stem augers or alternatively post-grouted wash bored anchors. The conventional earth anchors made in the clayey silt till deposit can be designed for a working bond adhesion of 50 to 60 kPa. It is expected that post-grouted anchors can be made in the native soils such that an anchor will safely carry about 60 kN/m of adhered anchor length (with a minimum diameter of 150 mm).

Where the excavation penetrates the bedrock, the rock excavation is nominally self-supporting in a vertical face, provided the rock bedding is horizontally oriented. Anchors made in bedrock of the Georgian Bay Formation may be designed using a working adhesion of 620 kPa.

The design adhesion for earth anchors is controlled as much by the installation technique as the soil and therefore a proto-type anchor must be made in each anchor level executed to demonstrate the anchor capacity and validate the design assumptions. A proto-type anchor must be made to demonstrate the anchor capacity (performance tested to 200% of the design load). All production anchors must be proof-tested to 133% of the design load, to validate the design assumptions.

Raker footings established on the sound bedrock at an inclination of 45 degrees can be designed for a maximum geotechnical resistance at ULS of 2,000 kPa.

4.9 Underpinning Considerations

Based on the anticipated depth of excavation and the distance from adjacent structures, the need for underpinning is not anticipated. However, the need for underpinning nearby structures will depend on the location of the structure and the respective foundation conditions. General guidelines are provided in Figure 5.

Consideration should be given to conduct a pre-construction condition survey of the adjacent buildings and infrastructure prior to commencing excavation. Underpinning recommendations could subsequently be provided upon request.

4.10 Site Work

The earth fill and native clayey silt till found at this site will become disturbed and may lose their integrity to support when subjected to traffic, particularly when wet. If there is site work carried out during periods of wet weather, then it can be expected that subgrade will be disturbed unless an adequate granular working surface is provided to protect the integrity of the subgrade soils from construction traffic. Subgrade preparation works cannot be adequately accomplished during wet weather and the project must be scheduled accordingly. The disturbance caused by the traffic can result in the removal of disturbed soil or bedrock and use of granular fill material or lean concrete mix for site restoration or underfloor fill that is not intrinsic to the project requirements, as required.

The most severe loading conditions on the subgrade may occur during construction. Consequently, special provisions such as end dumping and forward spreading of earth and aggregate fills, restricted construction lanes, and half-loads during placement of the granular base and other work may be required, especially if construction is carried out during unfavourable weather.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the founding subgrade and concrete must be provided. The soils at this site are susceptible to frost damage. Consideration must be given to frost effects, such as heave or softening, on exposed soil surfaces in the context of this particular project development.

4.11 Quality Control

The proposed structure will be founded on spread footing foundations. Our foundation recommendations are provided for foundations bearing directly on sound bedrock. As was previously indicated, rock coring was not carried out to determine the depth to sound bedrock, therefore the foundation installations must be field reviewed by Terraprobe as they are constructed. The on-site review of the condition of the foundation soil as the foundations are constructed, is an integral part of the geotechnical design function and is required by Section 4.2.2.2 of the Ontario Building Code (2012). If Terraprobe is not retained to carry out foundation evaluations during construction, then Terraprobe accepts no responsibility for the performance or non-performance of the foundations, even if they are ostensibly constructed in accordance with the design advice contained in this report.

The long-term performance of the slab on grade is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure that uniform subgrade moisture and density conditions are achieved as much as possible. The design advice in this report is based on an assessment of the subgrade support capabilities as indicated by the boreholes. These conditions may vary across the site depending on the final design grades and therefore, the preparation of the subgrade and compaction of all engineered fill should be monitored by Terraprobe at the time of construction to confirm material quality, thickness, and to ensure adequate compaction.

The requirements for fill placement on this project have been stipulated relative to Standard Proctor Maximum Dry Density (SPMDD). In situ determinations of density during fill placement on site are required to demonstrate that the specific placement density is achieved. Terraprobe is a CNSC certified operator of appropriate nuclear density gauges for this work and can provide sampling and testing services for the project as necessary, with our qualified technical staff.

Concrete will be specified in accordance with the requirements of CAN3 - CSA A23.1. Terraprobe maintains a CSA certified concrete laboratory and can provide concrete sampling and testing services for the project as necessary. Terraprobe staff can also provide quality control services for Building Envelope, Roofing and Structural Steel, as necessary, for the Structural and Architectural quality control requirements of the project. Terraprobe is certified by the Canadian Welding Bureau under W178.1-1996.

5.0 SLOPE STABILITY AND STREAMBANK EROSION ANALYSIS

The western portion of the project site abuts creek valley land, regulated by Credit Valley Conservation (CVC) Authority. Therefore, due to proximity of the valley slope, a slope stability and streambank erosion risk assessment is required to delineate the Long-Term Stable Slope Crest (LTSSC) location to help establish the extent of erosion hazard zone for the proposed development.

5.1 Visual Slope Inspection and Mapping

A detailed visual slope inspection of the slope area from the crest to the toe was conducted by Terraprobe on November 27, 2019. General information pertaining to the existing slope features such as slope profile, slope drainage, water course features, vegetation cover, buildings in the vicinity of the slope, erosion features, and slope slide features were noted during the inspection. A summary of the visual slope inspection is presented below. Photographs taken during the inspections are included as Appendix C. The locations of the features discussed below are shown on the Borehole Location and Site Features Plan provided as Figure 2.

The western portion of the project site (the existing parking lot) consists of the tableland abutting Mary Fix Creek, a tributary of Credit River. The tableland consists of an asphalt surfaced parking lot.

The valley slope within the study area is about 2.0 to 3.5 m high with inclination of about 1.5 to 2.0 horizontal to 1.0 vertical. A section of gabion retaining walls located along the southern portion of the slope provides the grade separation as well as erosion protection measure. The wall extends from a concrete box culvert at 2.5 m in height and tapers off northerly to 0 m in height for about 40 m in length. The gabion wall is in a fair condition.

Mary Fix Creek flows southerly at the slope toe and the bottom of the gabion wall. The creek is in direct contact with the slope toe. The creek bank was noted to be significantly vegetated and active erosion was not evident. Exposed bedrock is observed along the bank underlying the gabion wall, as well as the creek bed.

The slope face is well vegetated with trees and shrubs. The slope itself shows no evidence of instability or distress.

5.2 Slope Stability Analysis

Topographic information of the property and the slope (*Plan of Survey with Topography of Part of Block A, Registered Plan E-23, City of Mississauga, File No.: 190-0076, dated May 21, 2019, prepared by Speight, Van Nostran & Gibson Ltd*) was provided by the client and is enclosed (Figure 2). Four (4) representative cross sections (Sections A-A', B-B', C-C' and D-D') were inferred from the topographic information and our field observations to prepare slope model for the long-term slope stability analysis. The cross sections were selected on the basis of the slope height and inclination to provide adequate site coverage and represent the critical slope conditions present within the study area. These sections included a portion of the tableland extending across the slope down to the slope toe and beyond. The locations of the slope cross sections are presented on Figure 2, and the details of the slope profiles on Figures 6A and 6B (see enclosed).

A detailed engineering analysis of slope stability was carried out on the subject slope as shown in plan as Figure 3. The cross-sections used for the slope stability analysis carried out for this report were based on the provided survey data. The analysis was conducted utilizing computer software (Slide 2017 7.037, build date September 26, 2018, developed by Rocscience Inc.) and several standard methods of limit equilibrium analysis (Bishop, Janbu, Morgenstern/Price, and Spencer). These methods of analysis allow the calculation of Factors of Safety for hypothetical or assumed slip surfaces through the slope. The analysis method is used to assess potential for movements of large masses of soil over a specific slip surface which can be curved or circular, or non-circular. The analysis involves dividing the sliding mass into many thin slices and calculating the forces on each slice. The normal and shear forces acting on the sides and base of each slice are calculated. It is an iterative process that converges on a solution. An example analysis is provided as Appendix D, which shows the critical slip surface, the slices, and the inter-slice forces, as well as pertinent aspects of the slope stability output.

For a specific slip surface, the Factor of Safety is defined as the ratio of the available soil strength resisting movement, divided by the gravitational forces tending to cause movement. The Factor of Safety of 1.0 represents a “limiting equilibrium” condition where the slope is at a point of pending failure since the soil resistance is equal to forces tending to cause movement. It is usual to require a Factor of Safety greater than 1.0 to ensure stability of the slope. The typical Factor of Safety used for engineering design of slopes for stability ranges from about 1.3 to 1.5 for developments situated close to the slope crest. The most common design guidelines for “Active Land Use” are based on a 1.5 minimum Factor of Safety.

Each analysis was carried out by preparing a model of the slope geometry and subsurface conditions, and analyzing numerous different slip surfaces through the slope in search of the minimum or critical Factor of Safety for specific conditions. The pertinent data obtained from topographic plan, slope profiles, slope mapping, and the borehole information, were input for the slope stability analysis. Many calculations were carried out to examine the Factor of Safety for varying depths of potential slip surfaces. Circular and non-circular surfaces were both analyzed and circular surfaces were found to govern.

The average soil properties utilized for the soil strata in the slope stability analysis were assessed from factual information secured in each of the boreholes. The average soil properties are based on effective stress analysis for long-term slope stability, and are summarized in the table below. These soil properties are considered conservative; the soils on site are actually stronger. Short-term effects such as negative pore water pressures within unsaturated soils can increase the stability of a slope, and have been conservatively omitted.

Material	Unit Weight (kN/m ³)	Cohesion (kPa)	Internal Friction Angle (deg.)
Earth Fill	19	0	28
Clayey Silt Till	21	8	32
Gabion Wall	18	na*	na*
Clear Stone	20	0	32
Bedrock	26	na*	na*

* Bedrock and gabion wall modeled as a non-penetrable material with respect to potential soil slope slides

Based on the measurements from the ground water wells installed on site, the prevailing ground water table beneath the site is at approximate Elev. 110 ±m in the tableland. Conservatively, the potential effect of pore pressure on the long-term stability of the site slope was also assessed by incorporating an assumed elevated ground water level (located within about 1 to 2 m of the ground surface) to simulate infrequent and elevated ground water level (temporary condition) due to the potential seasonal fluctuation in the ground water table.

Slope cross sections (Sections A-A', B-B', C-C' and D-D') representing the critical slope conditions for the existing condition were analyzed. The results of the slope stability analysis of the existing conditions are provided in Appendix D, and are summarized in the table below.

Sections	Approximate Average Slope Inclination	Approximate Slope/Wall Height	Minimum Factor of Safety		Critical Slip Surface Description
			Normal Ground Water	Elevated Ground Water	
A-A'	2.0 H:1V	3.5 m	2.75	2.56	Circular slip surfaces pass through the overburden portion
B-B'	2.1H:1V	3.0 m	3.28	3.04	Circular slip surfaces pass through the overburden portion
C-C'	1.5 to 2.0 H:1V	2.5 m	1.51	1.45	Non-circular slip surfaces pass through the earth fill that likely composes the backfill and base material of the Gabion retaining wall
D-D'	N/A	2.5 m	1.71	1.61	Non-circular slip surfaces pass through the earth fill that likely composes the backfill and base material of the Gabion retaining wall

Circular surfaces were found to govern for the existing conditions at the analysed Sections A-A' and B-B', with critical slip surfaces passing through the entire slope profile. Non-circular surfaces governed for existing conditions analysed through the gabion retaining walls, Sections C-C' and D-D'. Critical failure paths were identified to pass through assumed details of the wall including backfill and base material. It

should be noted that the construction details of the gabion retaining wall are assumed and have not been confirmed.

Type	Land-Uses	Design Minimum Factor of Safety
A	PASSIVE: no buildings near slope; farm field, bush, forest, timberland, woods, wasteland, badlands, tundra	1.1
B	LIGHT: no habitable structures near slope; recreational parks, golf courses, buried small utilities, tile beds, barns, garages, swimming pools, sheds, satellite dishes, dog houses	1.20 to 1.30
C	ACTIVE: habitable or occupied structures near slopes; residential, commercial, and industrial buildings, retaining walls, storage/warehousing of non-hazardous substances	1.30 to 1.50
D	INFRASTRUCTURE and PUBLIC USE: public use structures and buildings (i.e. hospitals, schools, stadiums), cemeteries, bridges, high voltage power transmission lines, towers, storage/warehousing of hazardous materials, waste management areas	1.40 to 1.50

CVC *Slope Stability Definition & Determination Guideline* (Guideline) requires a minimum Factor of Safety of 1.5 for normal and 1.3 for the elevated temporary ground water condition.

The results of the analysis indicate that the existing slope and retaining wall has adequate Factor of Safety for both normal and elevated ground water conditions at each section. The relatively high Factors of Safety obtained for Sections A-A' and B-B' are representative of relatively thin overburden (approximately 3 m thickness) overlying the shale bedrock. However, as the creek flows southerly at the slope toe and the bottom of the gabion wall, the toe erosion allowance needs to be considered to determine the long-term slope crest location (discussed in Section 5.3). Therefore, additional analyses were carried out to determine the stable slope inclination (through overburden in the upper portion of the slope) for selected critical Section A-A' for both normal and elevated ground water level conditions. A number of representative trial profiles of the slope were analyzed to obtain a minimum factor of safety of 1.5 for normal and 1.3 for temporary and elevated ground water condition in conformance to the Guideline.

The results of the slope stability analysis conducted for a hypothetical slope profile with the inclination of 1.6 horizontal to 1.0 vertical for Section A-A' for both normal and elevated ground water conditions, are presented in Appendix D, and are summarized in the following table:

Sections	Approximate Average Slope Inclination	Approximate Slope Height	Minimum Factor of Safety		Critical Slip Surface Description
			Normal Ground Water	Elevated Ground Water	
A-A'	1.6 H:1V	3.5 m	1.98	1.96	Circular slip surfaces pass through the overburden portion

Based on the slope stability analysis, an inclination of 1.6 horizontal to 1.0 vertical (or flatter) for the upper overburden zone is required for the long-term stability at the site slope. The existing gabion retaining wall will be stable in the long-term provided that the routine inspection and maintenance is conducted.

5.3 Toe Erosion

In addition to the long-term stable slope inclination, a toe erosion allowance is applied in the determination of the LTSSC position (see Figure 7 for the general LTSSC model). The suggested design erosion allowances (Credit Valley Conservation Authority, *Watercourse & Valleyland Protection Policies*) are presented as follows.

Bank Condition Material At The Channel Bank Or Bank Full	Active Erosion of Bank	Erosion Currently Not Evident	Existing Erosion Protection In Place and Maintained along Bank
Limestone, Dolostone	2 m	1 m	0
Shale	5 m	2 m	0
Cohesive Soils; Silty Clays, Clayey Silt	8 m	4 m	0
Cohesionless Soils; Silts, Sands	15 m	7 m	0

The above table provides recommended toe erosion allowance based on the watercourse characteristics, type, and degree of slope toe erosion and the type of material comprised the slope toe. The CVC Guidelines recommend an erosion setback where the watercourse is located within 15 m of the slope toe. Mary Fix Creek at this site is located at the slope toe (within 15 m).

Exposed bedrock is observed along the creek bed and the lower sections of the slope profile. Active toe erosion was not observed along the east bank, which is adjacent to the subject property. The borehole data and site observations indicate that the slope toe material is expected to consist of shale bedrock. There is currently no evidence of active toe erosion, and therefore on the above considerations, a toe erosion allowance of 2 m is selected and applied at this site in conformance to the above CVC Guidelines table.

As previously noted, a section of gabion retaining walls located along the southern portion of the slope provides the erosion protection. The visual inspection indicates that this erosion protection measure is currently in a fair condition along the south side of the bank. Therefore, the toe erosion allowance would not be required along this length of the wall. Routine inspection and maintenance would be required for this protection measure.

5.4 Long-Term Stable Slope Crest (LTSSC) Position

The result of the global stability analysis indicates the existing slope at Sections A-A', B-B' and C-C' has adequate factor of safety against potential slope slides. However, it would not be stable in the long-term due to the potential toe erosion from the creek. An inclination of 1.6 horizontal to 1.0 vertical was applied to the overburden portion of the slope. The CVC Guideline recommends a stable slope inclination of 1.4

horizontal to 1 vertical for a shale slope without rock coring and characterization. Therefore, in conformance to the above, a stability setback of 1.4 horizontal to 1.0 vertical was applied to the shale portion of the slope at Sections A-A', B-B' and C-C'.

A toe erosion allowance of 2.0 m is applied to the valley slope at Sections A-A', B-B' and C-C' to account for potential toe erosion.

The result of the global stability analysis indicates the existing gabion retaining wall at Section D-D' has adequate factor of safety against potential slope slides.

The toe erosion allowance would not be required along this length of the wall. Routine inspection and maintenance would be required for this protection measure.

The location of the Long Term Stable Slope Crest (LTSSC) was determined based on the applicable stability and toe erosion setbacks in accordance with Long Term Stable Slope Crest Model (Figure 7). The LTSSC location is shown on Figure 2 in the plan and Figure 6 in the profile. For planning purposes, the long term refers to a 100 year planning horizon.

5.5 Development Setback / Erosion Access Allowance

In addition to the stability setback and the toe erosion allowance, the MNR and CVC Guideline require a Development Setback/Erosion Access Allowance to establish the Erosion Hazard Limit. The Erosion Hazard Limit consists of a combined setback based on the applicable stability, toe erosion and erosion access allowance. The policy guidelines require that the developments, dwellings, buildings or other structures should be further setback (erosion access allowance setback) from the greatest landward extent of the Physical Top of Bank, Staked Top of Bank and Long-Term Stable Slope Crest location. The erosion access allowance setback is usually required to facilitate access to the slope in case of an emergency/regular maintenance and to provide a buffer between the development and the valley system.

The erosion access setback requirements vary based on the policies and guidelines of individual authorities and site-specific conditions, and may vary, from 6 to 10 m based on MNR and individual Conservation Authority Guidelines. Structures may be allowed to be located closer if approved by applicable authorities and a qualified geotechnical engineer.

5.6 Slope Protection and Maintenance Considerations

The following general slope maintenance as well as construction considerations and constraints are recommended to maintain and enhance the slope condition, and to help protect against surficial soil erosion, during the development phase as well as in the long-term horizon:

1. Site development and construction activities should be conducted in a manner which does not result in surface erosion of the slope. In particular, site grading and drainage should be designed to prevent direct concentrated or channelized surface runoff from flowing directly over the slope. Water drainage from down-spouts, sumps, road drainage, and the like should not be permitted to flow over the slope, but a minor sheet flow may be acceptable. In case, tableland/downspout drainage is designed to be drained towards the slope, and approved by regulatory agencies, such drainage should be contained in a drainage pipe to convey flow directly and safely to the bottom of the slope.
2. The configuration of the slope should not be altered without prior consultation with a geotechnical engineer and conservation authority approval. In particular, the slope should not be steepened and fill materials/stockpiles should not be placed on the slope or within about 5 m of the slope crest.
3. A silt fence must be erected prior to the commencement of the site works and maintained until the completion of work or as required by the applicable authorities.
4. All necessary approvals must be secured from applicable authorities prior to the commencement of the site works.

It is recommended that the final site grading plans be reviewed by Terraprobe to ensure that they are consistent with the above recommendations.

6.0 LIMITATIONS AND USE OF REPORT

6.1 Procedures

The Terraprobe investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained from this investigation.

The drilling work was carried out by a drilling contractor and was observed and recorded by Terraprobe on a full time basis. All boreholes were made by a continuous flight power auger machine using solid stem augers, with the ability to record SPT-N values. A Terraprobe technician logged the boreholes and examined the samples as they were obtained. The samples obtained were sealed in clean, air-tight containers and transferred to the Terraprobe laboratory, where they were reviewed for consistency of description by a geotechnical engineer. Ground water observations were made in the boreholes as drilling proceeded.

The samples of the strata penetrated were obtained using the Split-Barrel Method technique (ASTM D1586). The samples were taken at intervals. The conventional interval sampling procedure used for this investigation does not recover continuous samples of soil at any borehole location. There is consequently some interpolation of the borehole layering between samples and indications of changes in stratigraphy as shown on the borehole logs are approximate.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. A comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations.

It may not be possible to drill a sufficient number of boreholes, or sample and report them in a way that would provide all the subsurface information and geotechnical advice to completely identify all aspects of the site and works that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project must be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, and their approach to the construction works, cognizant of the risks implicit in the subsurface investigation activities.

6.2 Changes in Site and Scope

The passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. In particular, caution should be exercised in the consideration of contractual responsibilities as they relate to control of seepage, disturbance of soils, and frost protection.

The design parameters provided and the engineering advice offered in this report are based on the factual data obtained from this investigation made at the site by Terraprobe and are intended for use by the owner and its retained design consultants in the design phase of the project. If there are changes to the project scope and development features, the interpretations made of the subsurface information, the geotechnical design parameters, advice and comments relating to constructability issues and quality control may not be relevant or complete for the project. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report.

6.3 Use of Report

This report is prepared for the express use of Ranee Management, and their retained design consultants. It is not for use by others. This report is copyright of Terraprobe Inc., and no part of this report may be reproduced by any means, in any form, without the prior written permission of Terraprobe.

Ranee Management and their retained design consultants are authorized users.

It is recognized that the City of Mississauga, in its capacity as the planning and building authority under Provincial statutes, will make use of and rely upon this report, cognizant of the limitations thereof, both as are expressed and implied.

We trust the foregoing information is sufficient for your present for your present requirements. If you have any questions, or if we can be of further assistance, please do not hesitate to contact us.

Terraprobe Inc.



B. V. Jayabaskaran
B. V. Jayabaskaran, P.Eng.
Geotechnical Engineer

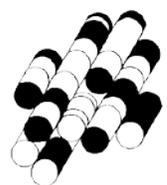
A handwritten signature in black ink, appearing to read "Seth Zhang".

Seth Zhang, M.Eng., M.Sc., P.Eng.
Associate



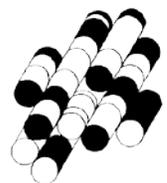
ENCLOSURES

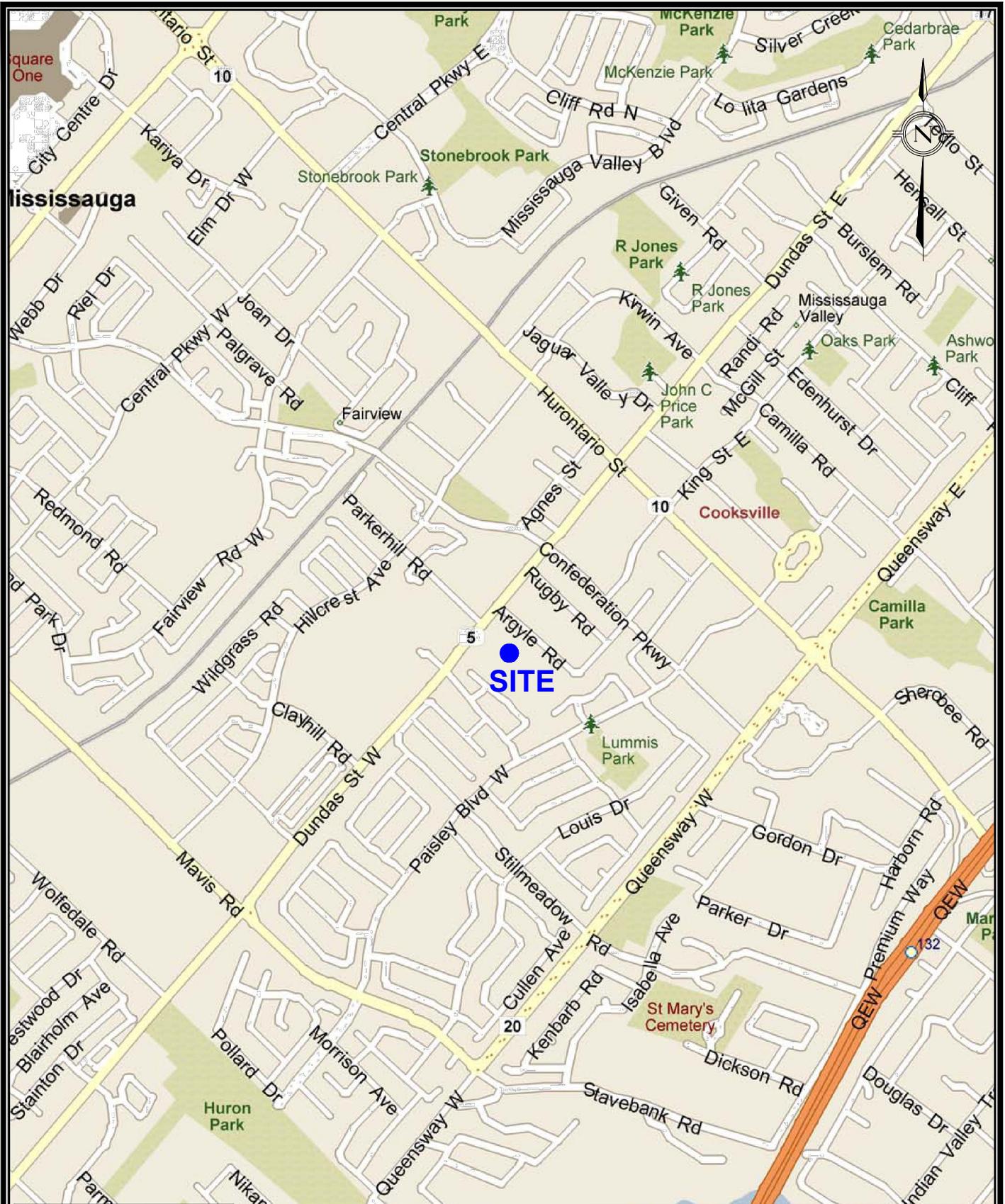
TERRAPROBE INC.



FIGURES

TERRAPROBE INC.





REFERENCE
Microsoft Streets & Trips 2012



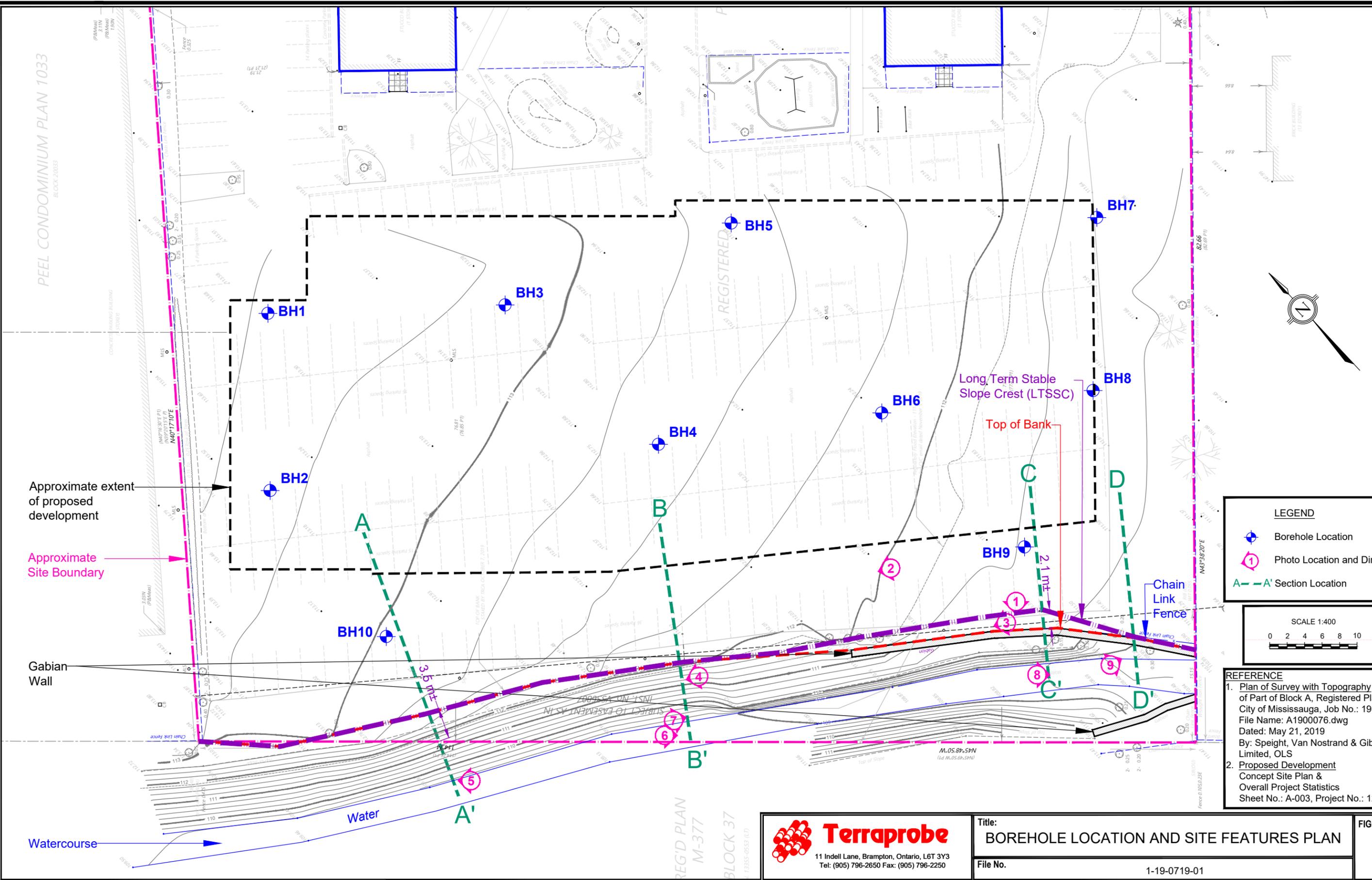
Title: **SITE LOCATION PLAN**

File No. 1-19-0719-01

FIGURE :
1

Z:\1-Project-Files\2019\1-19-0719-01-2570 - 2590-Argyle Road, Mississauga\01-560- Investigation\A. Dwyer\Logs\AurCAD\1-19-0719-01 FIG 1.dwg, Sandy

PEEL CONDOMINIUM PLAN 1033
BLOCK 20033



Approximate extent of proposed development

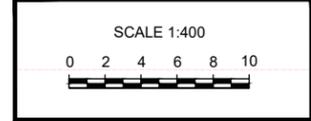
Approximate Site Boundary

Gabian Wall

Watercourse

LEGEND

- Borehole Location
- Photo Location and Direction
- A-A' Section Location



- REFERENCE**
1. Plan of Survey with Topography of Part of Block A, Registered Plan E-23 City of Mississauga, Job No.: 190-0076 File Name: A1900076.dwg Dated: May 21, 2019 By: Speight, Van Nostrand & Gibson Limited, OLS
 2. Proposed Development Concept Site Plan & Overall Project Statistics Sheet No.: A-003, Project No.: 120325

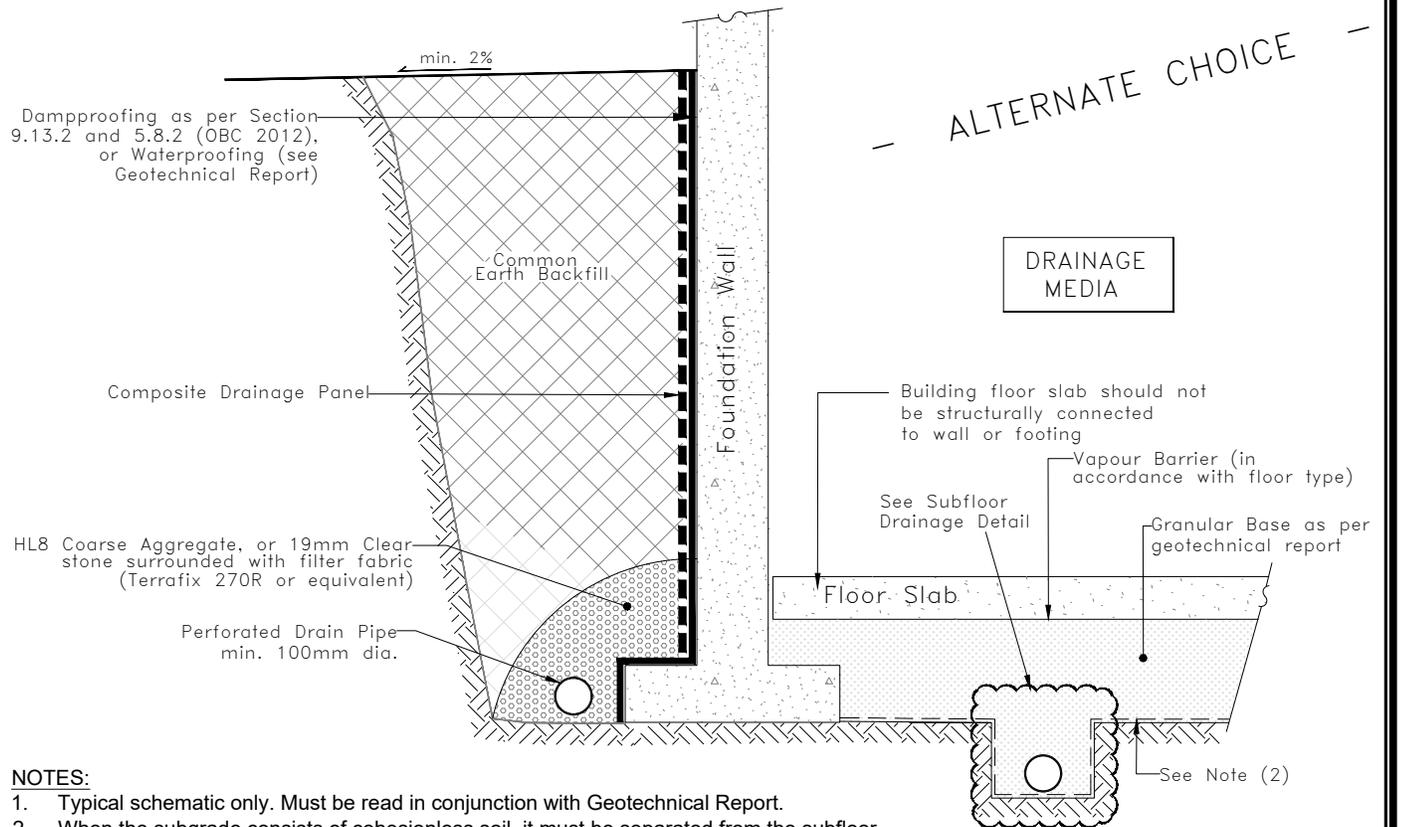
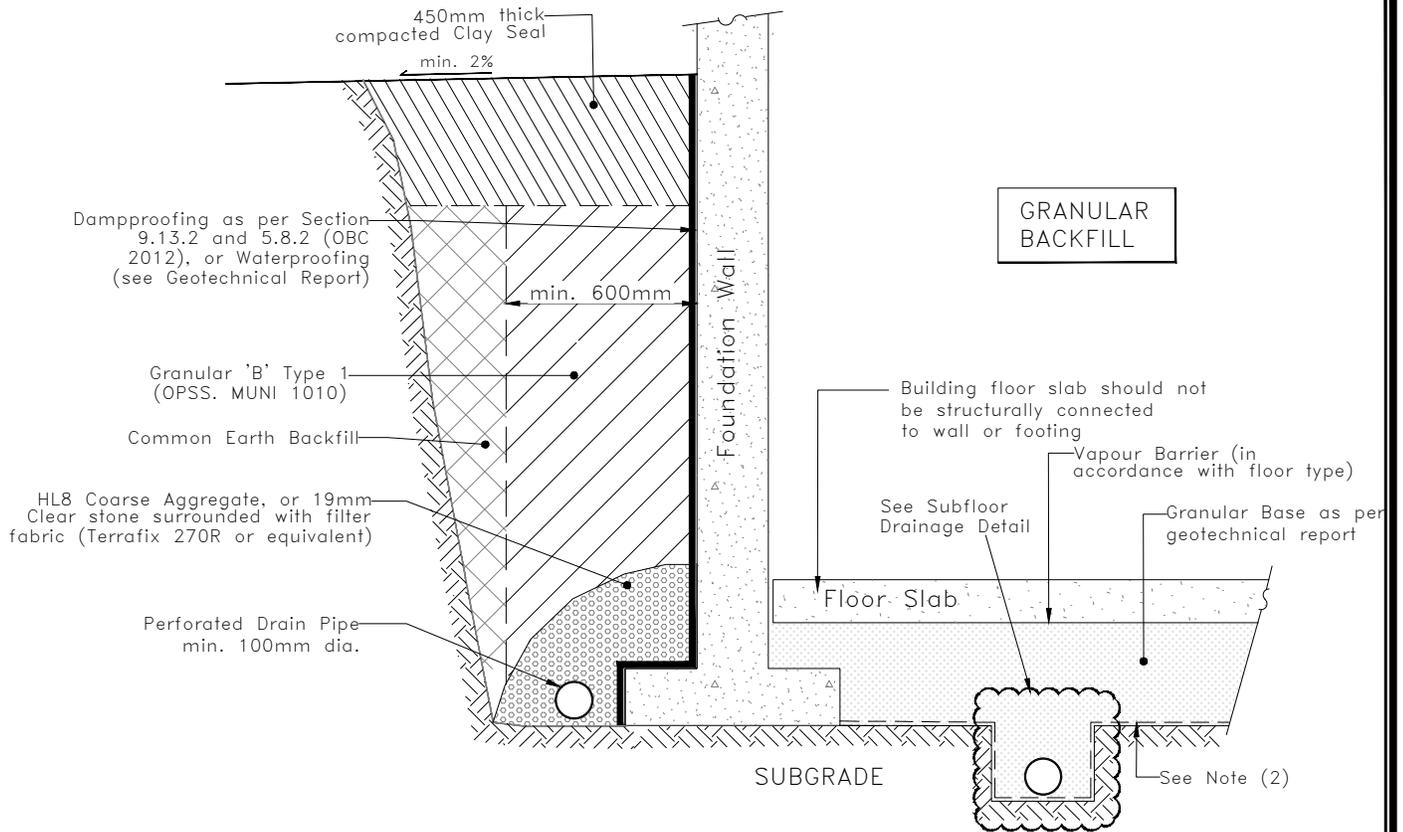
Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title: **BOREHOLE LOCATION AND SITE FEATURES PLAN**

File No. 1-19-0719-01

FIGURE:
2

\\192.168.1.16\sa1\Project Files\2019\1-19-0719-01-2570-2590 Anylye Road, Mississauga\01-ECO Investigation\A_Dwg_Loop\AurCAD\1-19-0719-01_2020-05-29_RIG.dwg
Kamal

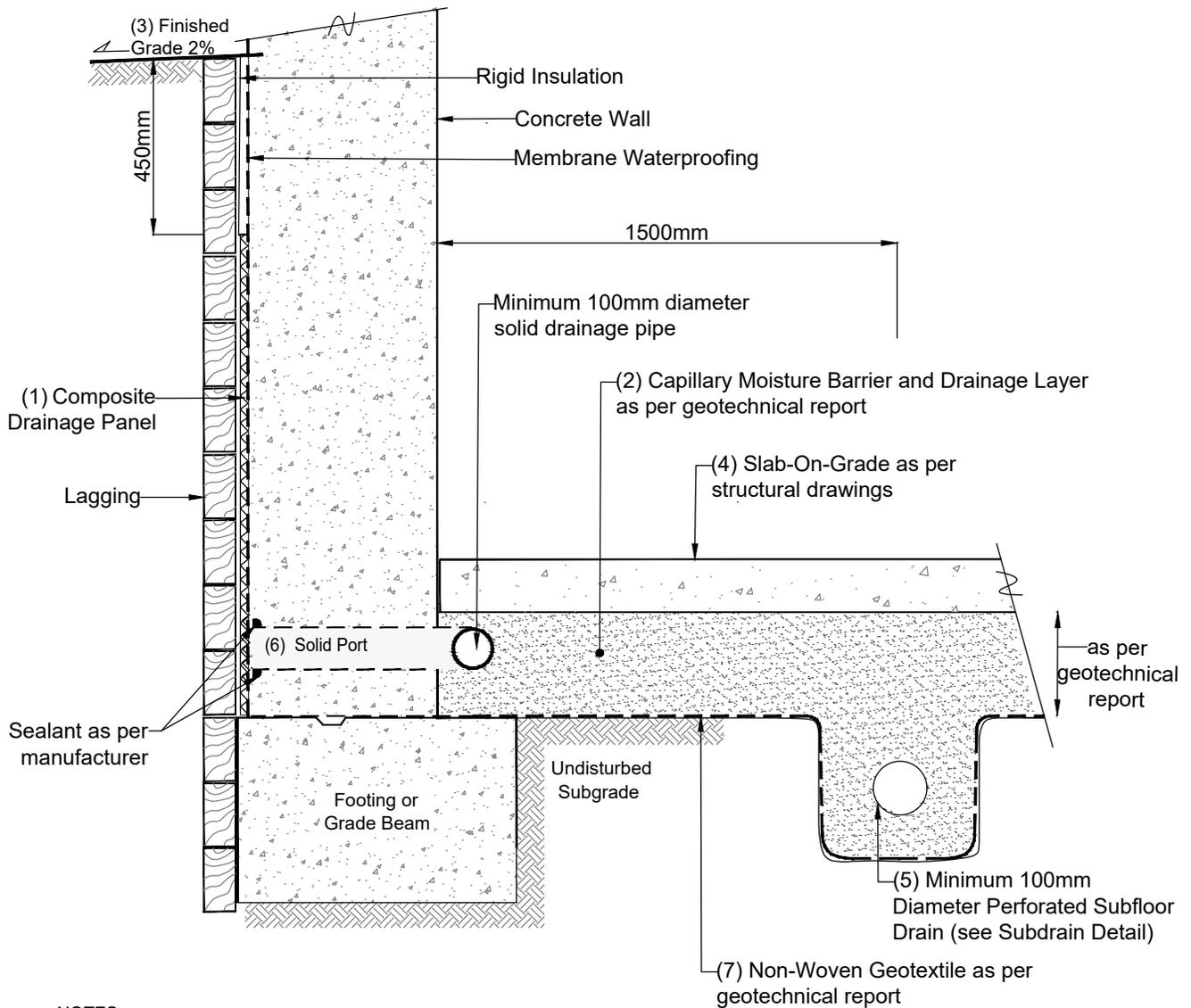


NOTES:

1. Typical schematic only. Must be read in conjunction with Geotechnical Report.
2. When the subgrade consists of cohesionless soil, it must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).
3. Not to Scale

Title:

**TYPICAL BASEMENT DRAINAGE SCHEMATIC
(OPEN EXCAVATION)**



NOTES

- 1) Prefabricated composite drainage panels to consist of Miradrain 6000, or approved equivalent. Panels should provide continuous cover as per manufacturer's requirements.
- 2) Capillary moisture barrier/drainage layer to consist of a minimum 200mm layer of 19mm clear stone (OPSS. MUNI 1004), or as indicated in geotechnical report, compacted to a dense state. Upper 50mm can be replaced with Granular "A" (OPSS. MUNI 1010) compacted to 98% SPMDD where vehicular traffic is required. A vapour barrier may be required depending on floor type.
- 3) Exterior finished grade away from wall at a minimum grade of 2% for min. 1.2m.
- 4) Building floor slab-on-grade shall not be structurally connected to foundation wall or footing.
- 5) Subfloor drain invert to be a minimum of 300mm below underside of floor slab, to be set in parallel rows, one way, and at the spacing specified in the geotechnical report. Don't connect subfloor drains to perimeter drains.
- 6) Embedded ports to be set a distance of maximum 3m on-centre. Each port to have a minimum cross-sectional area of 1500mm². Perimeter drainage must be collected and conveyed directly to the building sumps in solidpipe.
- 7) When the subgrade consists of a cohesionless soil, the subgrade must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).
- 8) Geotechnical report contains specific details. Final detail must be reviewed before system is considered acceptable to use.

N.T.S.

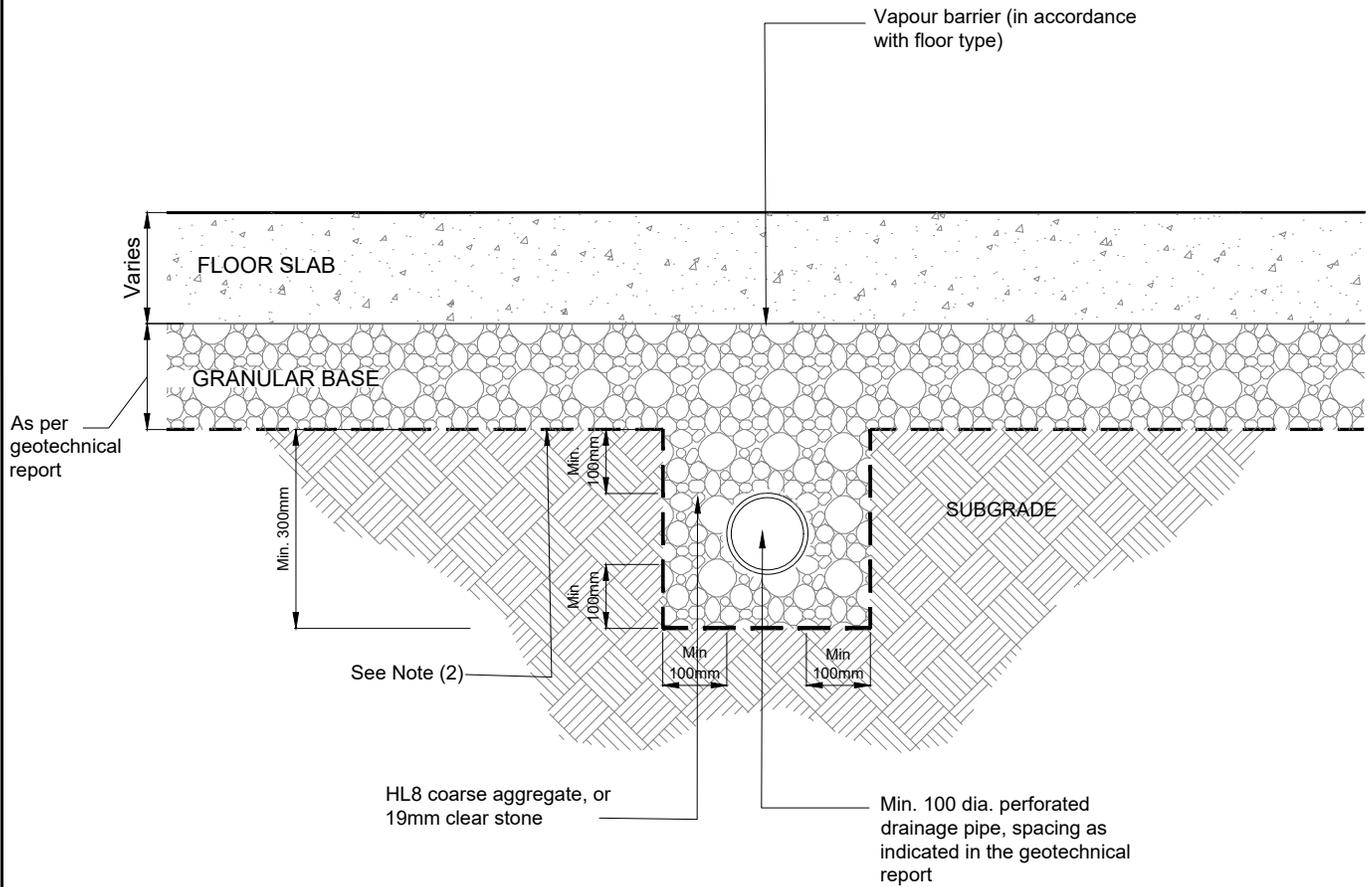


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Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

**SCHEMATIC DRAINAGE DETAIL
SOLDIER PILE & LAGGING SHORING SYSTEM
(ONE-SIDED WALL CONSTRUCTION)**



NOTES:

1. Typical schematic only. Must be read in conjunction with Geotechnical Report.
2. When the subgrade consists of cohesionless soil, it must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).
3. Not to Scale

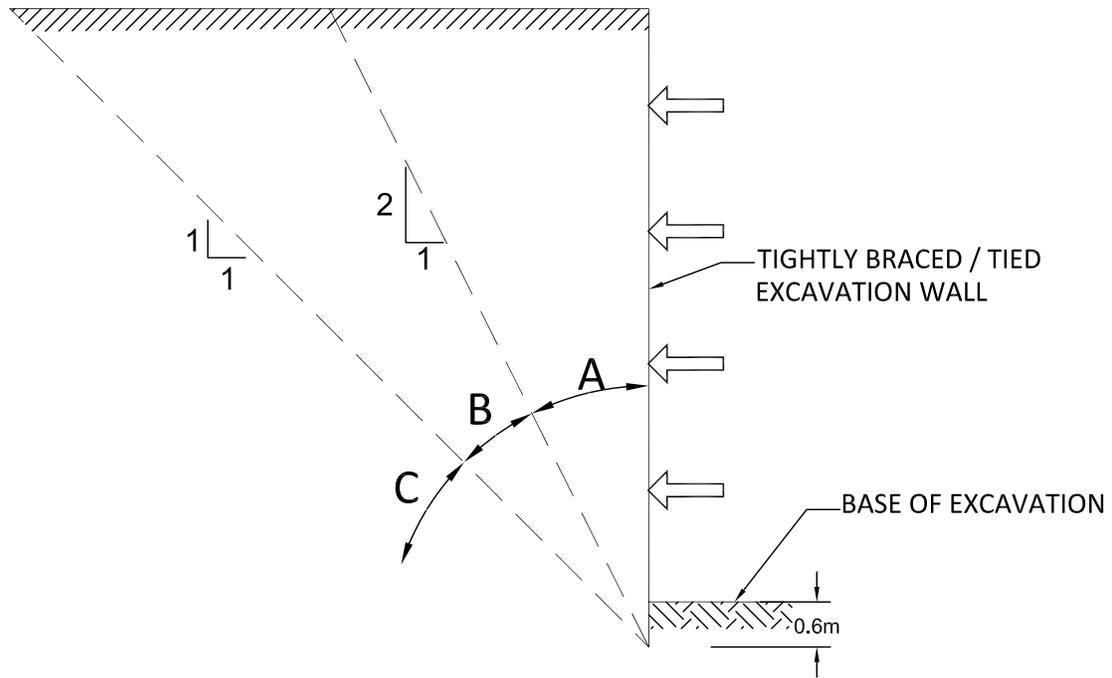


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Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

TYPICAL BASEMENT SUBDRAIN DETAIL



Zone A: Foundations within this zone often require underpinning. Horizontal and vertical pressures on excavation wall of non-underpinned foundations must be considered.

Zone B: Foundation within this zone often do not require underpinning. Horizontal and vertical pressures on excavation wall of non-underpinned foundations must be considered.

Zone C: Foundations within this zone usually do not require underpinning.

REFERENCE:

User's Guide - NBC 2005 Structural Commentaries
(Part 4 of Division B) - Commentary K



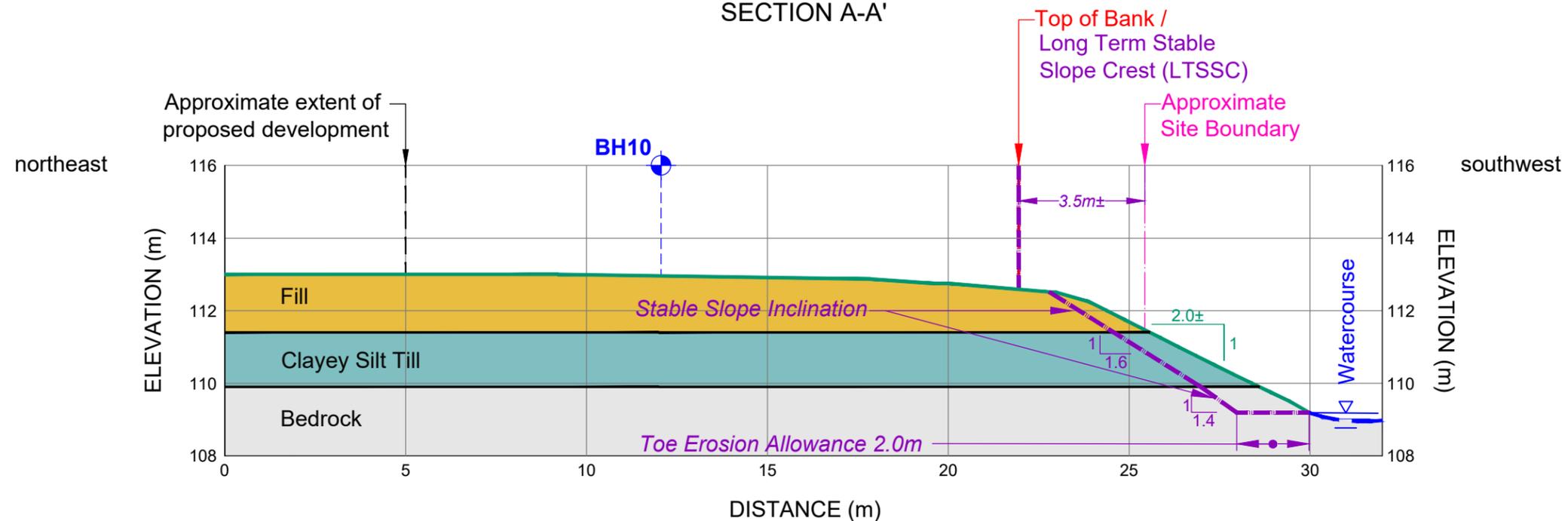
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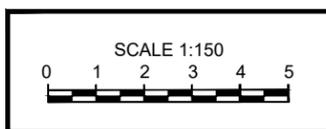
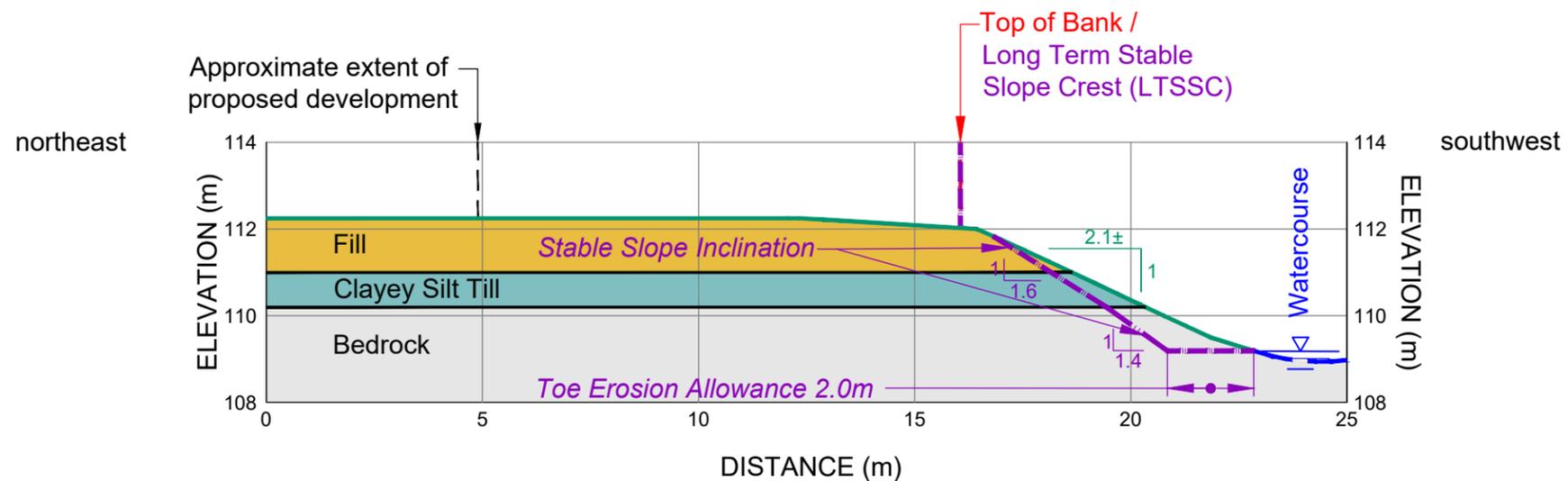
Title:

GUIDELINES FOR UNDERPINNING SOILS

SECTION A-A'



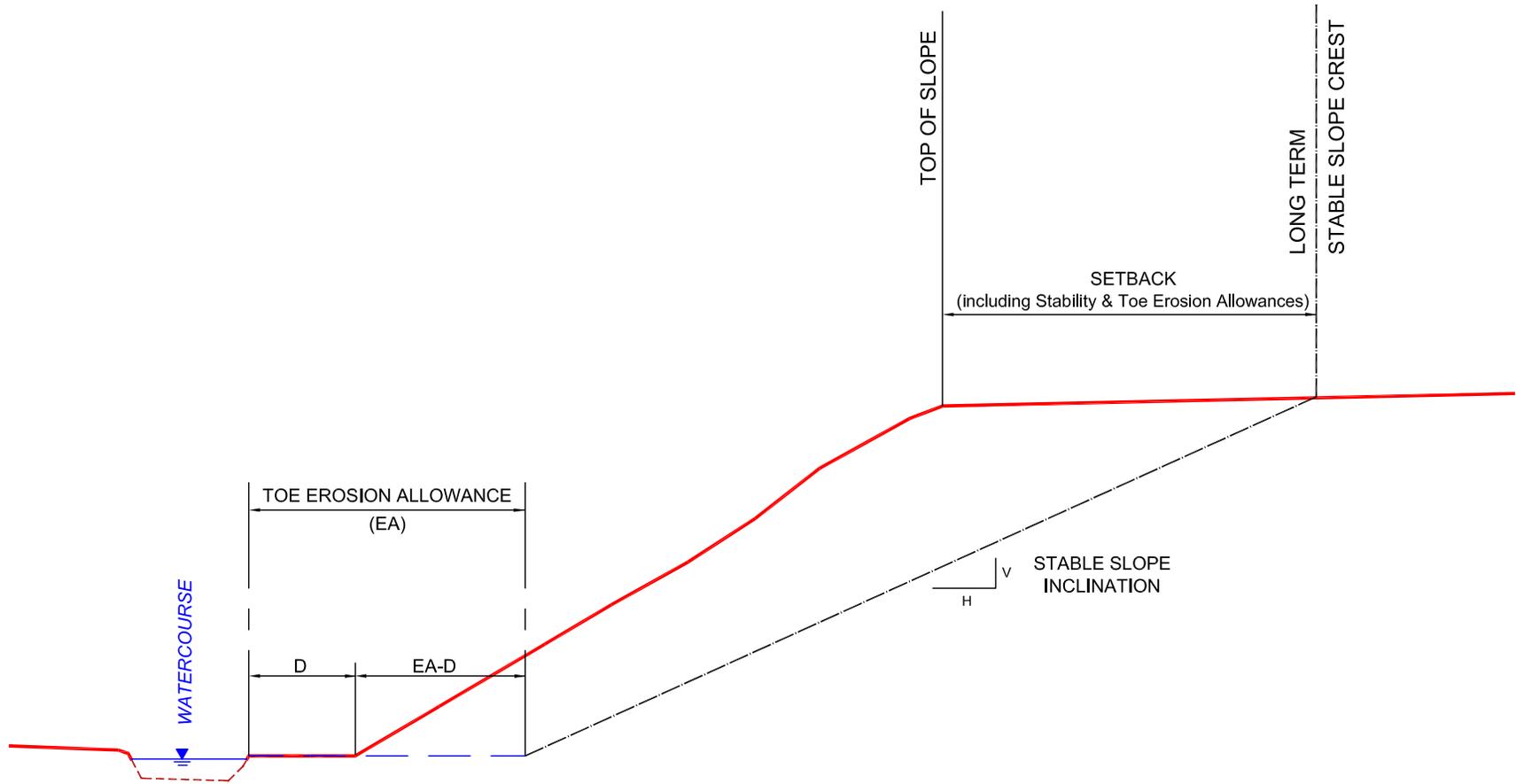
SECTION B-B'



Title:	SLOPE CROSS SECTIONS
File No.	1-19-0719-01

FIGURE :
6A

\\192.168.1.16\sa1\Project Files\2019\1-19-0719-01-19-0719-01 (2020-05-29) RIG.dwg
Kamal



LEGEND

D = Available Flood Plain
Between Edge of Watercourse and
Slope Toe

EA = Erosion Allowance



Terraprobe

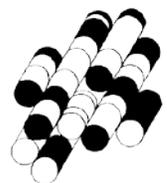
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

LONG TERM STABLE SLOPE CREST MODEL

APPENDIX A

TERRAPROBE INC.





SAMPLING METHODS		PENETRATION RESISTANCE
AS	auger sample	<p>Standard Penetration Test (SPT) resistance ('N' values) is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a standard 50 mm (2 in.) diameter split spoon sampler for a distance of 0.3 m (12 in.).</p> <p>Dynamic Cone Test (DCT) resistance is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a conical steel point of 50 mm (2 in.) diameter and with 60° sides on 'A' size drill rods for a distance of 0.3 m (12 in.)."</p>
CORE	cored sample	
DP	direct push	
FV	field vane	
GS	grab sample	
SS	split spoon	
ST	shelby tube	
WS	wash sample	

COHESIONLESS SOILS		COHESIVE SOILS			COMPOSITION	
Compactness	'N' value	Consistency	'N' value	Undrained Shear Strength (kPa)	Term (e.g)	% by weight
very loose	< 4	very soft	< 2	< 12	<i>trace</i> silt	< 10
loose	4 – 10	soft	2 – 4	12 – 25	<i>some</i> silt	10 – 20
compact	10 – 30	firm	4 – 8	25 – 50	silty	20 – 35
dense	30 – 50	stiff	8 – 15	50 – 100	sand <i>and</i> silt	> 35
very dense	> 50	very stiff	15 – 30	100 – 200		
		hard	> 30	> 200		

TESTS AND SYMBOLS

MH	mechanical sieve and hydrometer analysis		Unstabilized water level
w, w _c	water content		1 st water level measurement
w _L , LL	liquid limit		2 nd water level measurement
w _P , PL	plastic limit		Most recent water level measurement
I _P , PI	plasticity index		
k	coefficient of permeability		
γ	soil unit weight, bulk		
G _s	specific gravity		
φ'	internal friction angle		
c'	effective cohesion		
c _u	undrained shear strength		
		3.0+	Undrained shear strength from field vane (with sensitivity)
		C _c	compression index
		c _v	coefficient of consolidation
		m _v	coefficient of compressibility
		e	void ratio

FIELD MOISTURE DESCRIPTIONS

Damp	refers to a soil sample that does not exhibit any observable pore water from field/hand inspection.
Moist	refers to a soil sample that exhibits evidence of existing pore water (e.g. sample feels cool, cohesive soil is at or close to plastic limit) but does not have visible pore water
Wet	refers to a soil sample that has visible pore water

Project No. : 1-19-0719-01

Client : Ranee Management

Originated by : DH

Date started : December 3, 2019

Project : 2570 - 2590 Argyle Road

Compiled by : BV

Sheet No. : 1 of 1

Location : Mississauga, Ontario

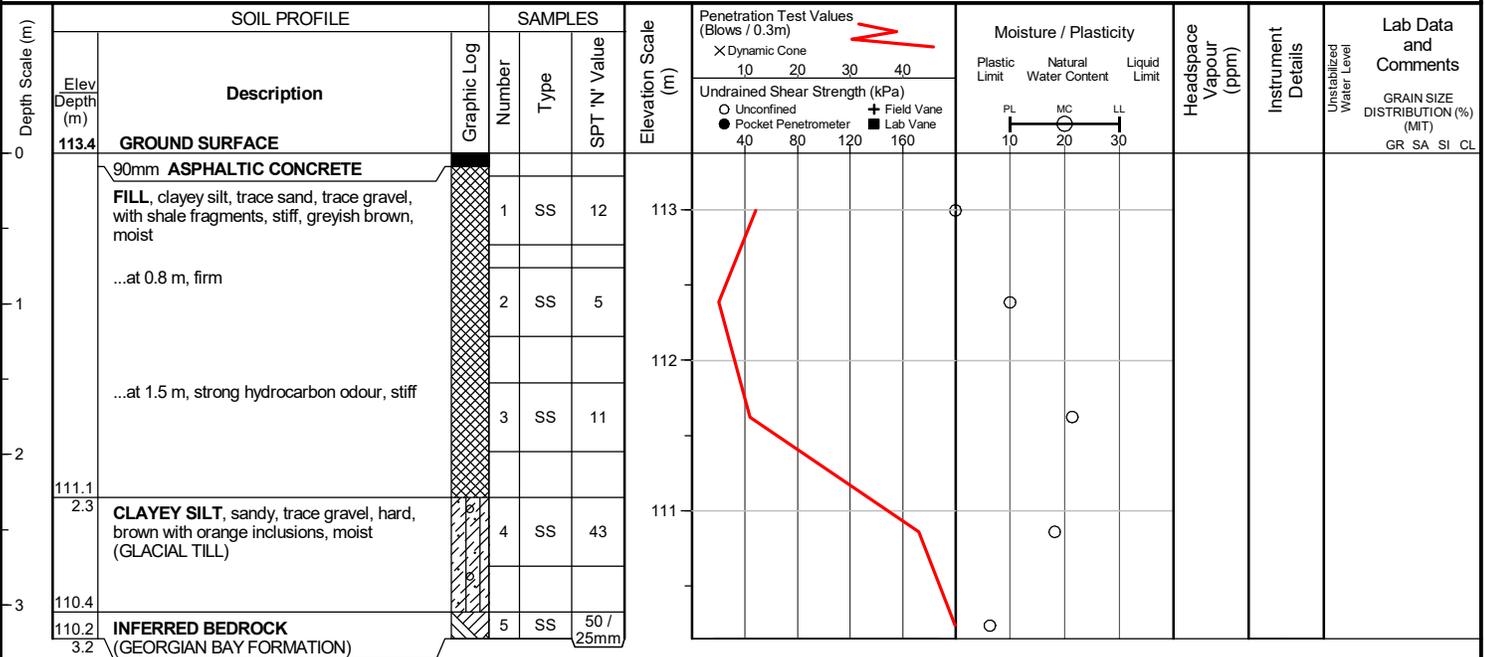
Checked by : SZ

Position : E: 611285, N: 4825513 (UTM 17T)

Elevation Datum : Geodetic

Rig type : B-37

Drilling Method : Solid stem augers


END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

Project No. : 1-19-0719-01

Client : Ranee Management

Originated by : DH

Date started : December 3, 2019

Project : 2570 - 2590 Argyle Road

Compiled by : BV

Sheet No. : 1 of 1

Location : Mississauga, Ontario

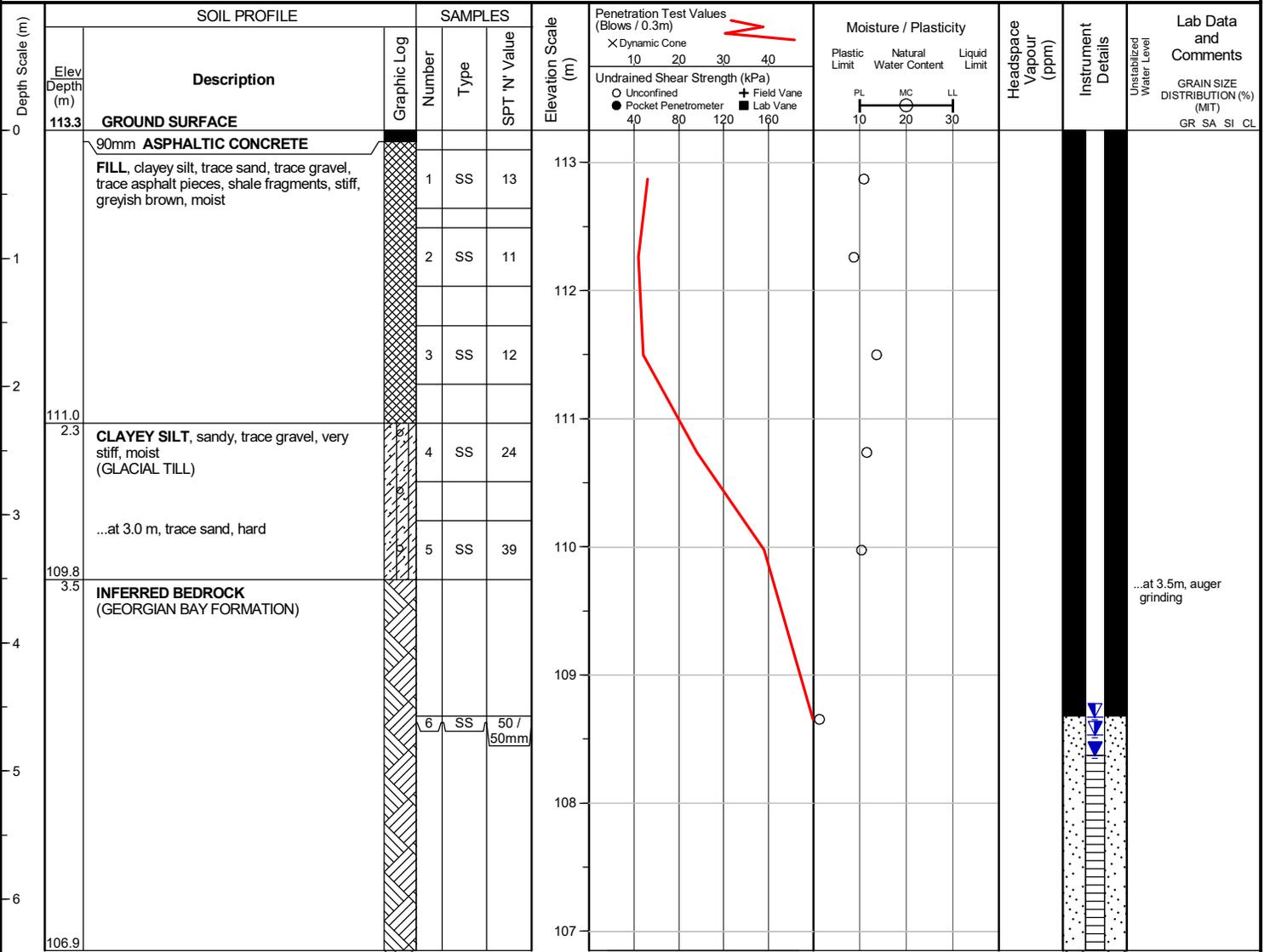
Checked by : SZ

Position : E: 611271, N: 4825498 (UTM 17T)

Elevation Datum : Geodetic

Rig type : B-37

Drilling Method : Solid stem augers


END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Dec 10, 2019	4.6	108.7
Dec 23, 2019	4.7	108.5
Jan 9, 2020	4.9	108.4

Project No. : 1-19-0719-01

Client : Ranee Management

Originated by : DH

Date started : December 3, 2019

Project : 2570 - 2590 Argyle Road

Compiled by : BV

Sheet No. : 1 of 1

Location : Mississauga, Ontario

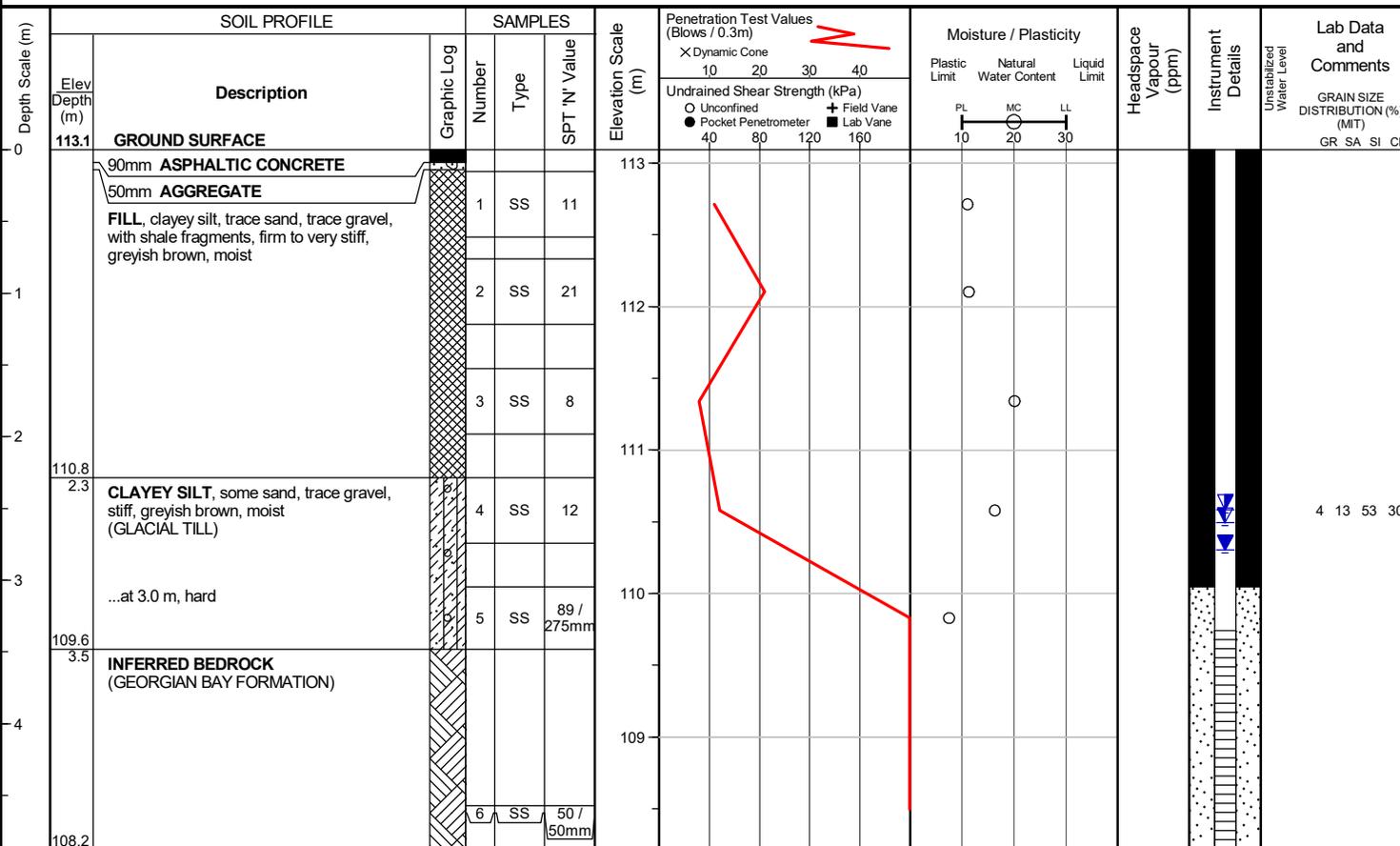
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Position : E: 611305, N: 4825495 (UTM 17T)

Elevation Datum : Geodetic

Rig type : B-37

Drilling Method : Solid stem augers


END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Dec 10, 2019	2.6	110.5
Dec 23, 2019	2.5	110.6
Jan 9, 2020	2.8	110.3

Project No. : 1-19-0719-01

Client : Ranee Management

Originated by : DH

Date started : December 2, 2019

Project : 2570 - 2590 Argyle Road

Compiled by : BV

Sheet No. : 1 of 1

Location : Mississauga, Ontario

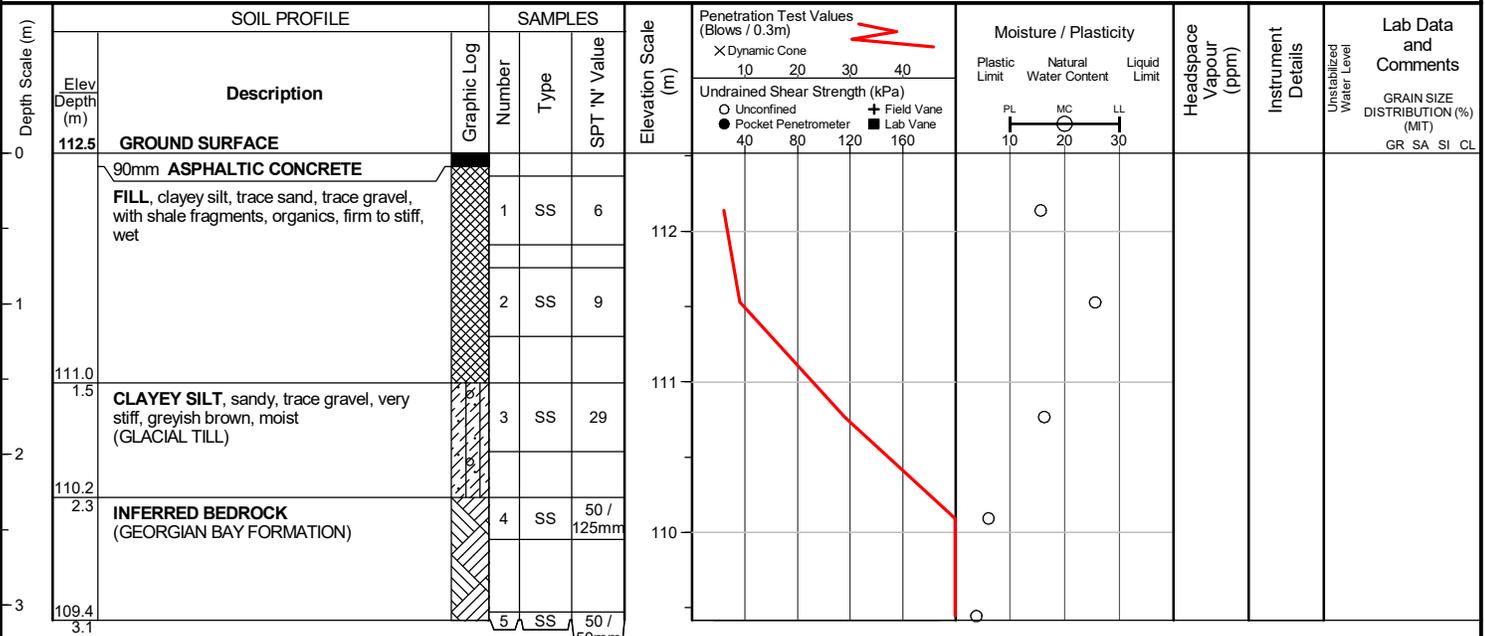
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Position : E: 611306, N: 4825471 (UTM 17T)

Elevation Datum : Geodetic

Rig type : B-37

Drilling Method : Solid stem augers



Borehole was dry and open upon completion of drilling.

Project No. : 1-19-0719-01

Client : Ranee Management

Originated by : DH

Date started : December 2, 2019

Project : 2570 - 2590 Argyle Road

Compiled by : BV

Sheet No. : 1 of 1

Location : Mississauga, Ontario

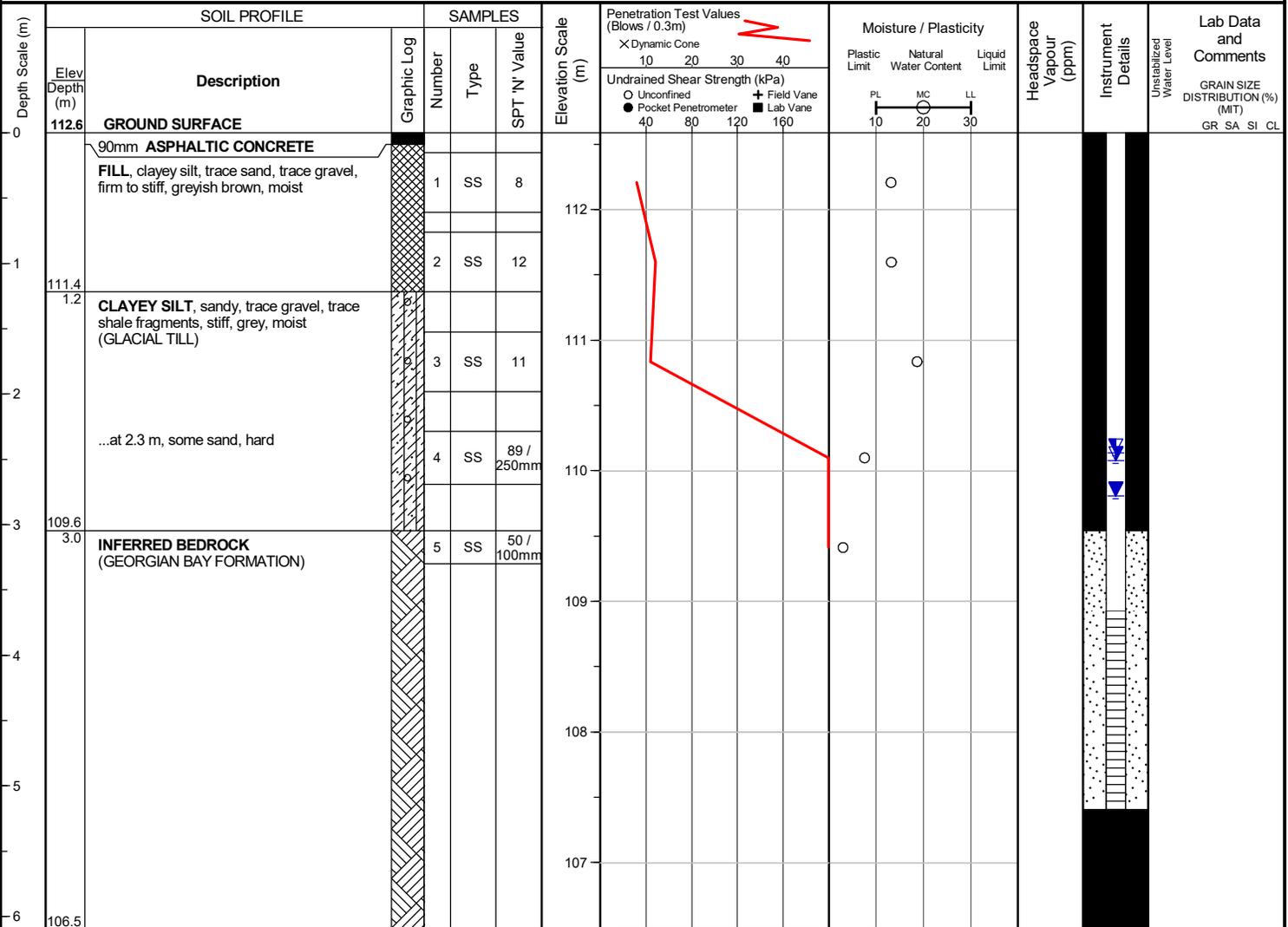
Checked by : SZ

Position : E: 611330, N: 4825483 (UTM 17T)

Elevation Datum : Geodetic

Rig type : B-37

Drilling Method : Solid stem augers


END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Dec 10, 2019	2.5	110.1
Dec 23, 2019	2.5	110.1
Jan 9, 2020	2.8	109.8

Project No. : 1-19-0719-01

Client : Ranee Management

Originated by : DH

Date started : December 2, 2019

Project : 2570 - 2590 Argyle Road

Compiled by : BV

Sheet No. : 1 of 1

Location : Mississauga, Ontario

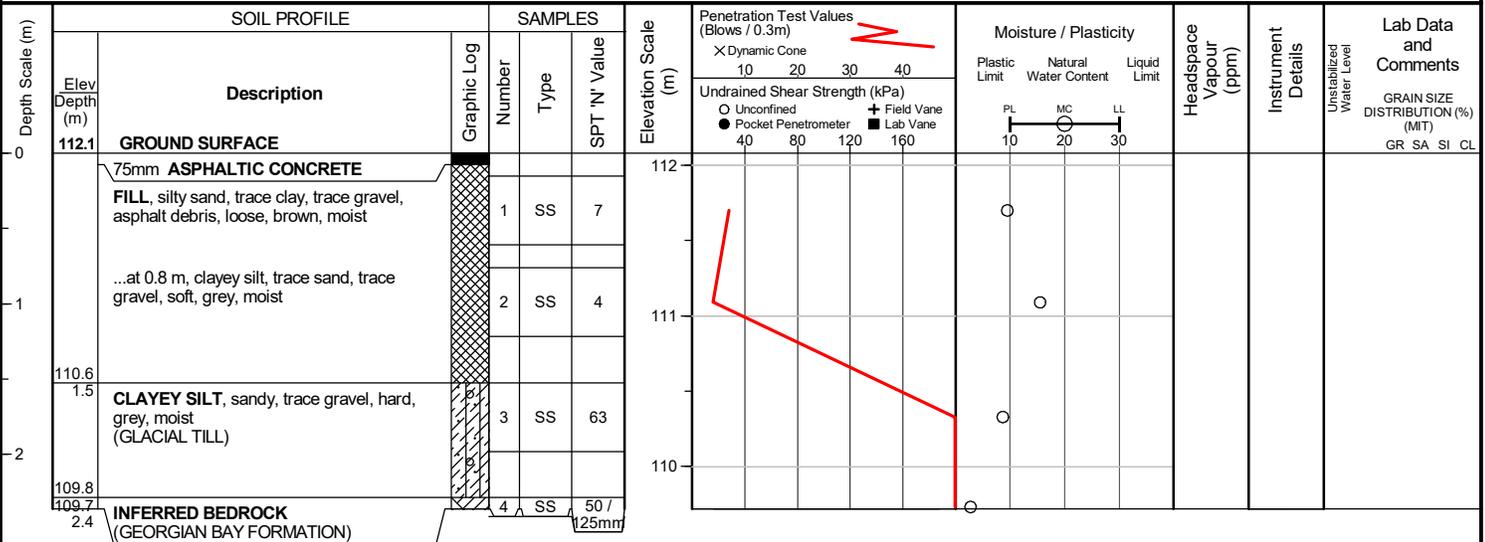
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Position : E: 611327, N: 4825456 (UTM 17T)

Elevation Datum : Geodetic

Rig type : B-37

Drilling Method : Solid stem augers


END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

Project No. : 1-19-0719-01

Client : Ranee Management

Originated by : DH

Date started : December 2, 2019

Project : 2570 - 2590 Argyle Road

Compiled by : BV

Sheet No. : 1 of 1

Location : Mississauga, Ontario

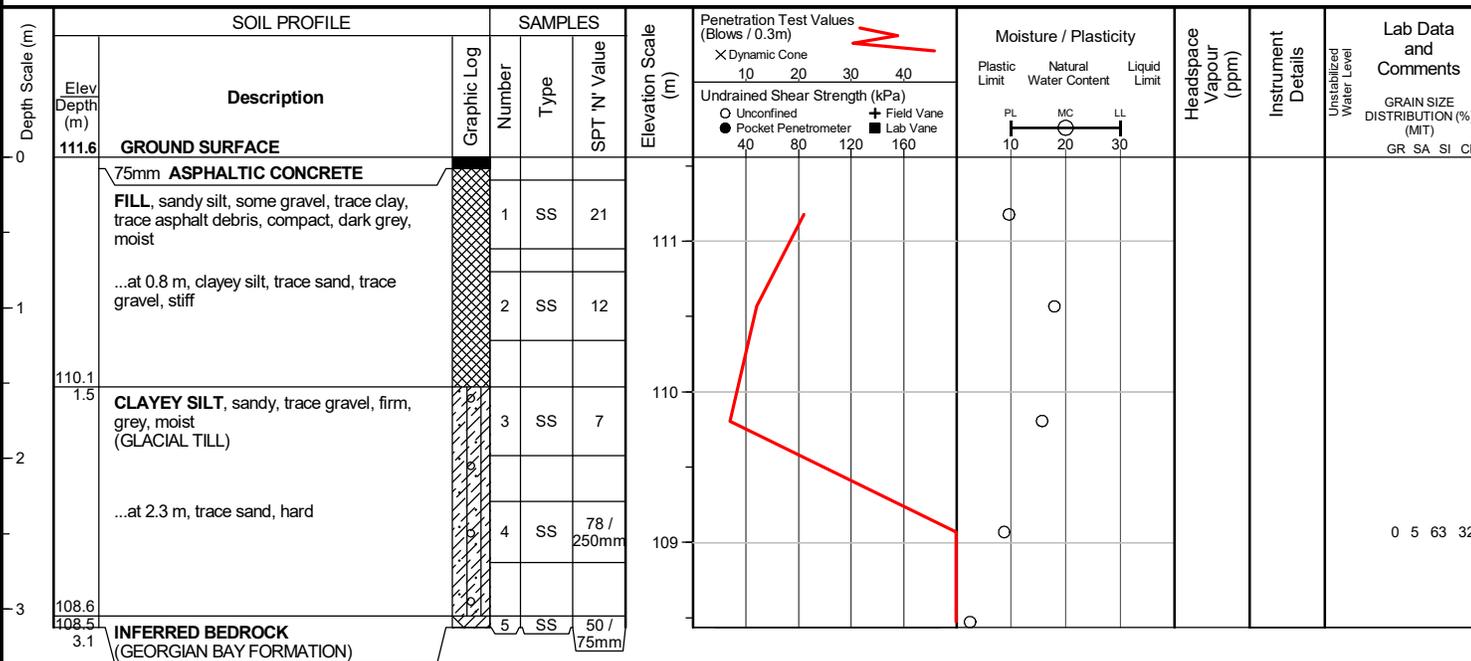
Checked by : SZ

Position : E: 611360, N: 4825455 (UTM 17T)

Elevation Datum : Geodetic

Rig type : B-37

Drilling Method : Solid stem augers


END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

Project No. : 1-19-0719-01

Client : Ranee Management

Originated by : DH

Date started : December 4, 2019

Project : 2570 - 2590 Argyle Road

Compiled by : BV

Sheet No. : 1 of 1

Location : Mississauga, Ontario

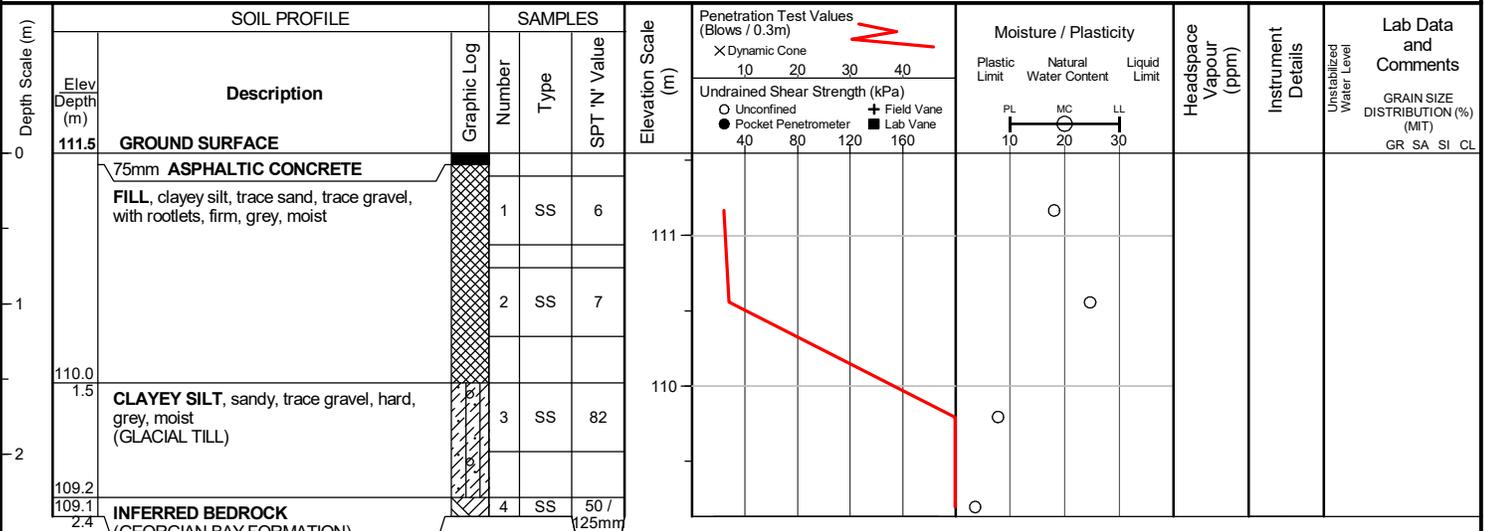
Checked by : SZ

Position : E: 611346, N: 4825441 (UTM 17T)

Elevation Datum : Geodetic

Rig type : B-37

Drilling Method : Solid stem augers



END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

Project No. : 1-19-0719-01

Client : Ranee Management

Originated by : DH

Date started : December 4, 2019

Project : 2570 - 2590 Argyle Road

Compiled by : BV

Sheet No. : 1 of 1

Location : Mississauga, Ontario

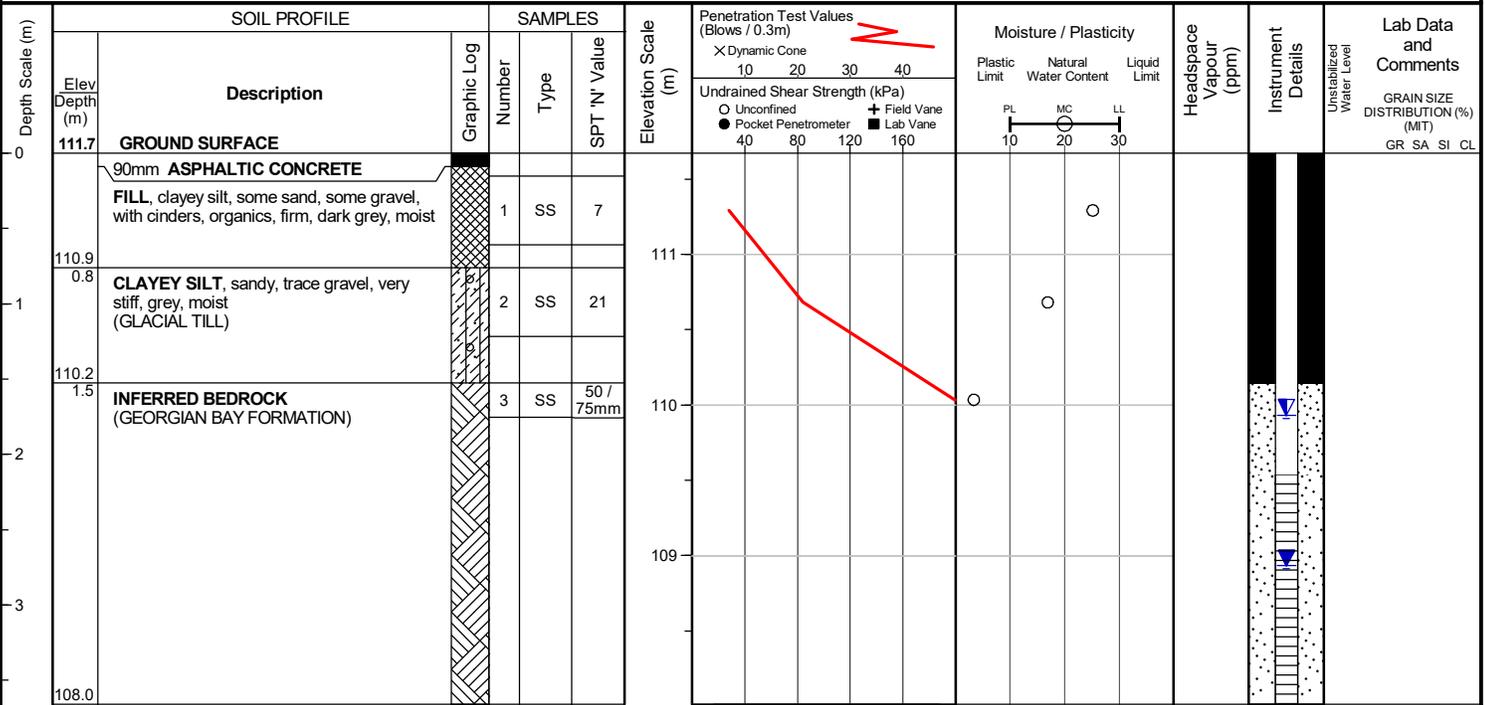
Checked by : SZ

Position : E: 611328, N: 4825433 (UTM 17T)

Elevation Datum : Geodetic

Rig type : B-37

Drilling Method : Solid stem augers


END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Dec 10, 2019	1.7	109.9
Dec 23, 2019	n/a	n/a
Jan 9, 2020	2.7	108.9

Project No. : 1-19-0719-01 Client : Ranee Management Originated by : DH
 Date started : December 3, 2019 Project : 2570 - 2590 Argyle Road Compiled by : BV
 Sheet No. : 1 of 1 Location : Mississauga, Ontario Checked by : SZ

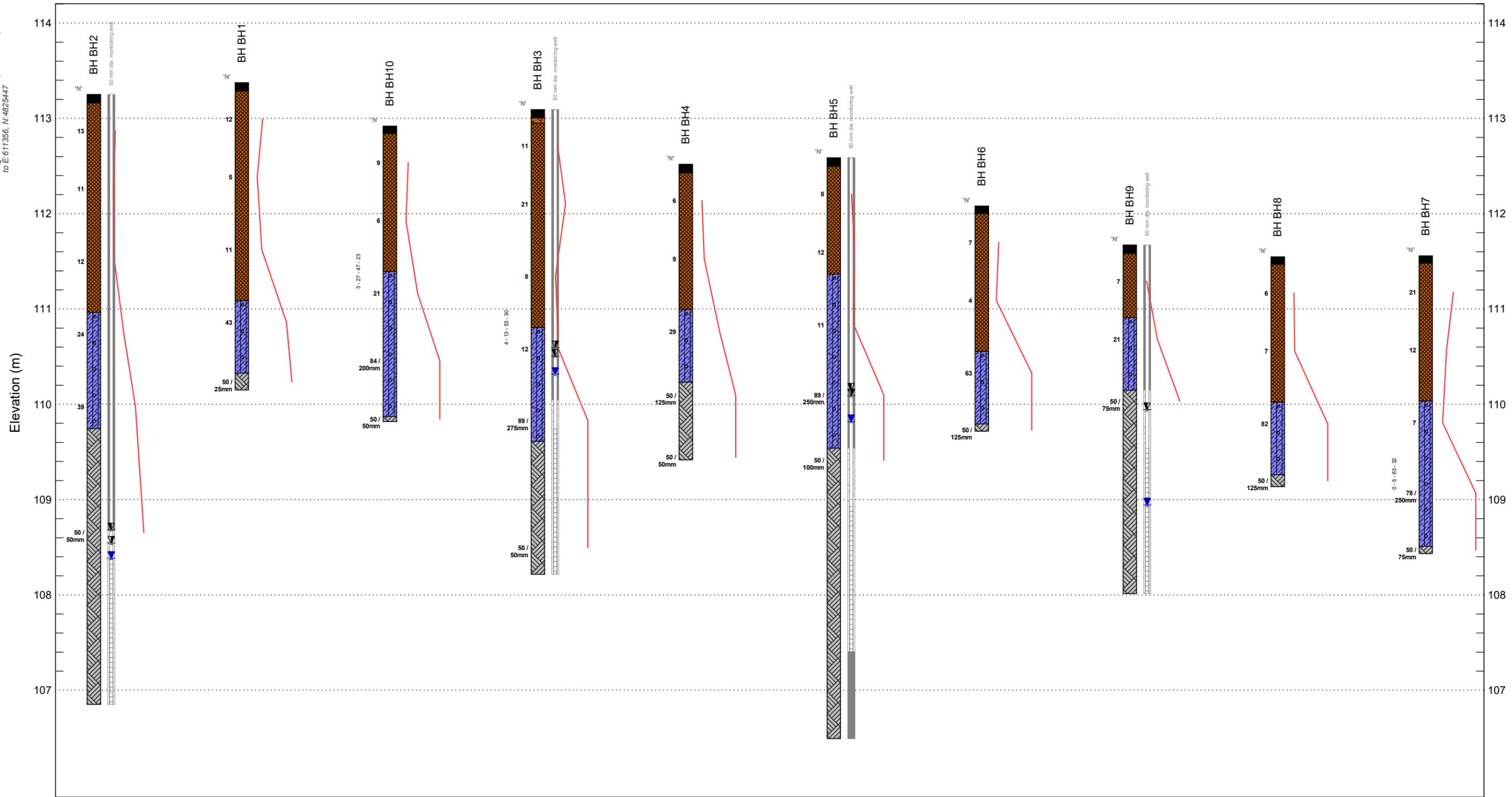
Position : E: 611269, N: 4825477 (UTM 17T) Elevation Datum : Geodetic
 Rig type : B-37 Drilling Method : Solid stem augers

Depth Scale (m)	SOIL PROFILE		SAMPLES			Elevation Scale (m)	Penetration Test Values (Blows / 0.3m) X Dynamic Cone 10 20 30 40 Undrained Shear Strength (kPa) 40 80 120 160 ○ Unconfined ● Pocket Penetrometer + Field Vane ■ Lab Vane	Moisture / Plasticity			Headspace Vapour (ppm)	Instrument Details	Lab Data and Comments Unstabilized Water Level GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
	Elev Depth (m)	Description	Graphic Log	Number	Type			SPT 'N' Value	Plastic Limit	Natural Water Content			
0	112.9	GROUND SURFACE											
		75mm ASPHALTIC CONCRETE											
		FILL , silty sand, trace gravel, trace clay, with brick debris, asphalt concrete, loose, brown, moist ...at 0.8 m, clayey silt, firm, dark grey, wet		1	SS	9							
				2	SS	6							
	111.4	CLAYEY SILT , sandy, trace gravel, very stiff, greyish brown, moist (GLACIAL TILL) ...at 2.3 m, trace sand, hard, grey, damp		3	SS	21							3 27 47 23
	1.5			4	SS	84 / 200mm							
	109.9	INFERRED BEDROCK (GEORGIAN BAY FORMATION)		5	SS	50 / 50mm							
	109.8												
	3.1												

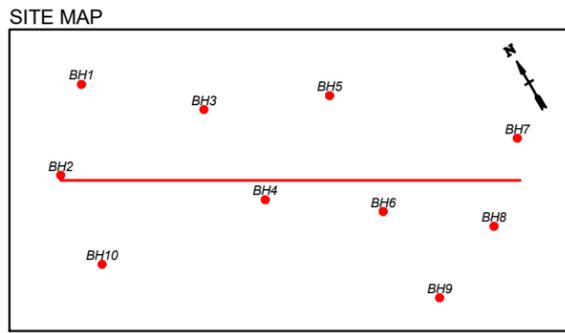
END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

Alignment: From E:611270, N:4825497,
to E:611356, N:4825447



Boreholes Equally Spaced



LITHOLOGY GRAPHIC LEGEND

- Asphalt
- Aggregate
- Fill
- Clayey Silt Till
- Bedrock

INTERPRETIVE LEGEND

- WL on completion of drilling
- Stabilized WL, most recent

- FILL
- GRAVELS (gravel to gravelly sand)
- SILT TO SAND (not till)
- COHESIONLESS TILLS
- COHESIVE SOILS (clayey silt to clay, incl. tills)
- DISTURBED/REWORKED SOILS

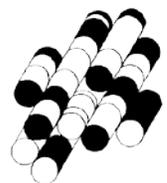
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

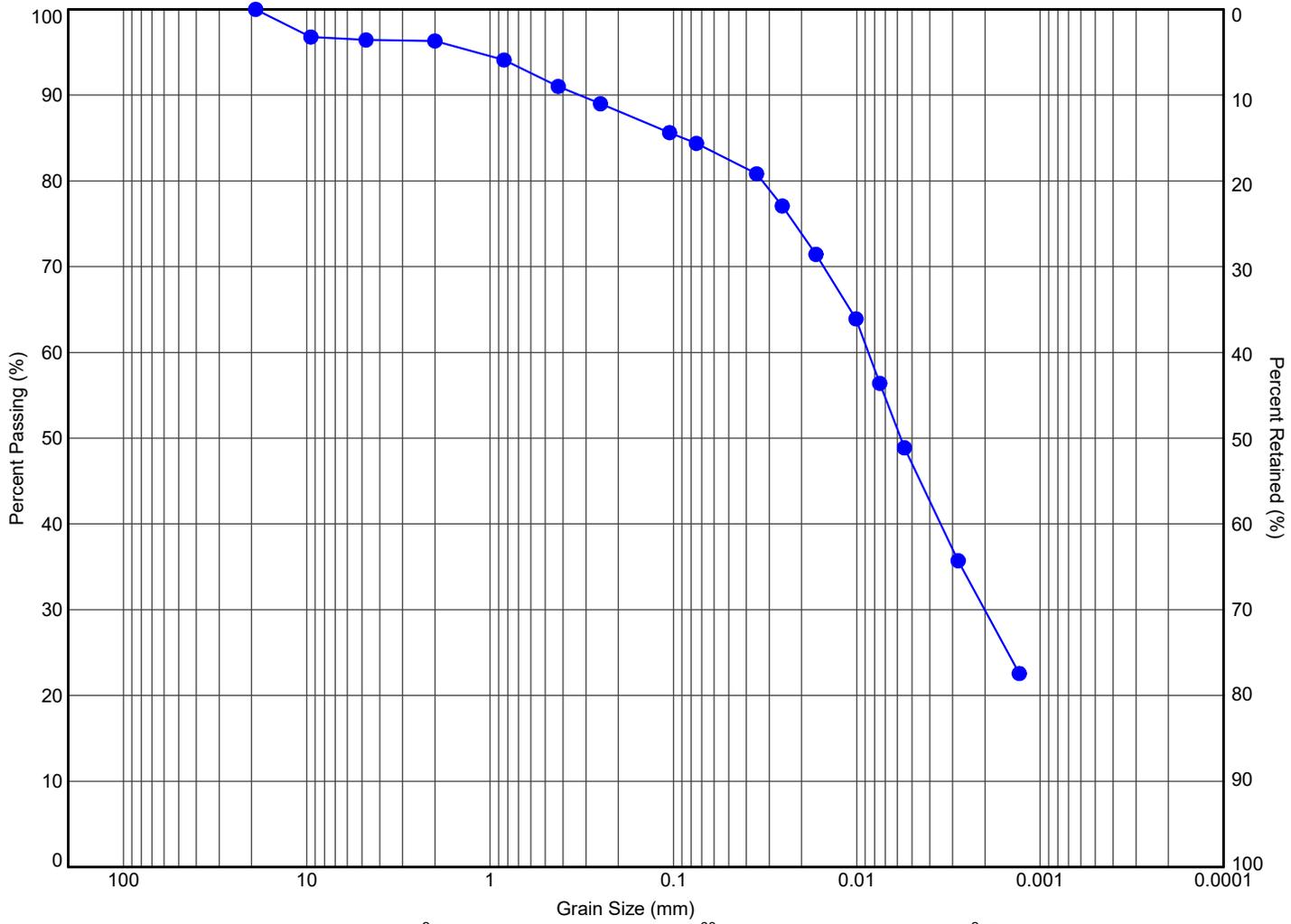
Title:	SUBSURFACE PROFILE
File No.:	1-19-0719-01

Report: ISECTION - TABLOID - ELEV

APPENDIX B

TERRAPROBE INC.





MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

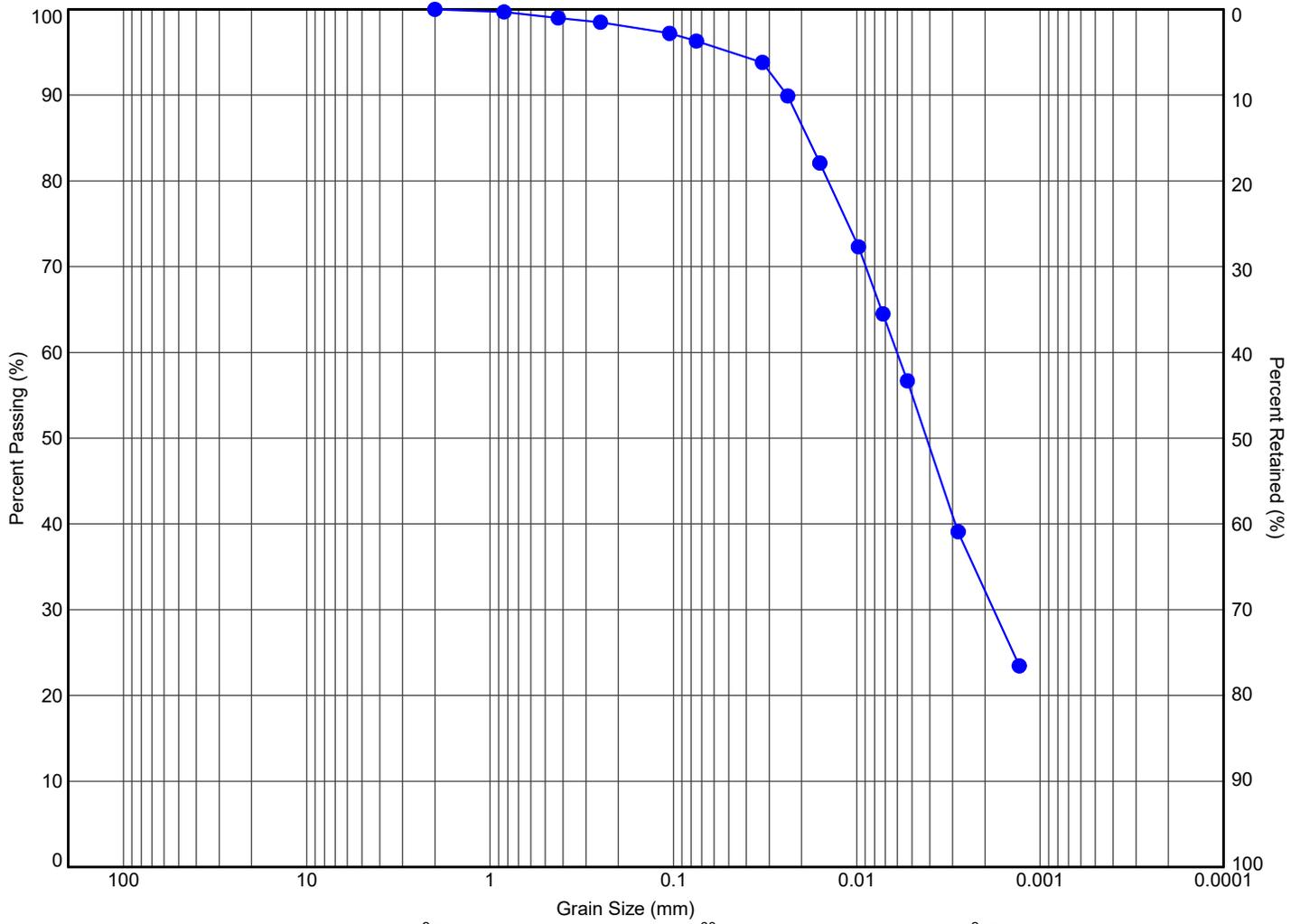
MIT SYSTEM									
Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	(Fines, %)	
● BH3	SS4	2.5	110.6	4	13	53	30		



11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title: **GRAIN SIZE DISTRIBUTION**
CLAYEY SILT, SOME SAND, TRACE GRAVEL

File No.: **1-19-0719-01**



MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

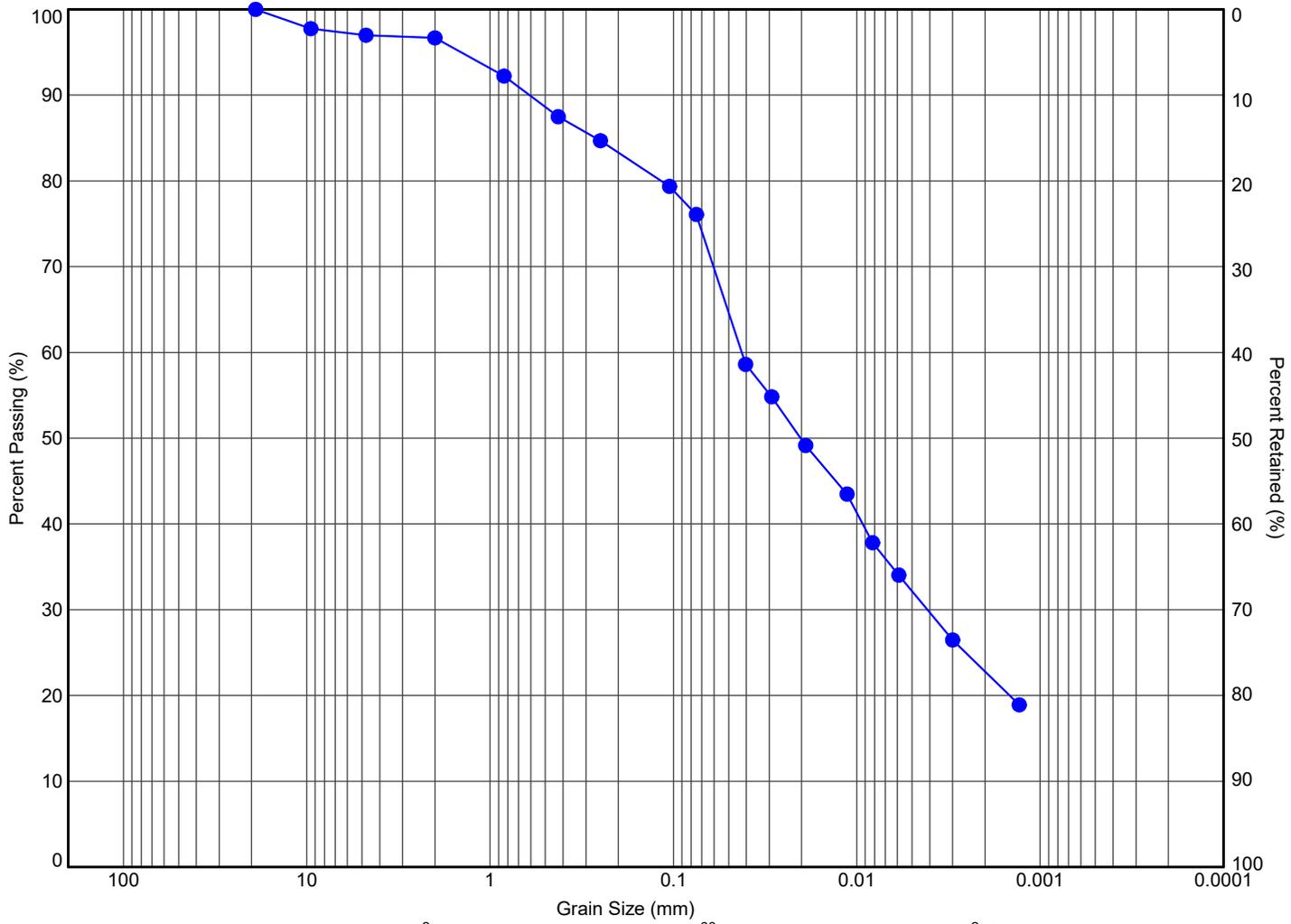
MIT SYSTEM									
Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	(Fines, %)	
● BH7	SS4	2.5	109.1	0	5	63	32		



11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title: **GRAIN SIZE DISTRIBUTION
CLAYEY SILT, TRACE SAND**

File No.: **1-19-0719-01**



MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

MIT SYSTEM

Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	(Fines, %)
● BH10	SS3	1.8	111.2	3	27	47	23	



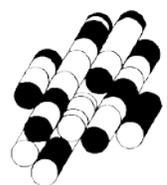
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title: **GRAIN SIZE DISTRIBUTION
CLAYEY SILT, SANDY, TRACE GRAVEL**

File No.: **1-19-0719-01**

APPENDIX C

TERRAPROBE INC.





Photograph 1

Location: Tableland
Viewing: Southwest, towards slope crest
Description: Flat parking lot/tableland, dense vegetation visible on slope face.



Photograph 2

Location: Tableland
Viewing: Northwest
Description: Relatively flat tableland.



Photograph 3

Location: Slope Crest near Section C-C'
Viewing: Northwest
Description: Dense vegetation, trunk of growth is mostly near vertical. Refuse material visible on slope face.





Photograph 4

Location: Slope Crest near Section B-b'
Viewing: Northwest
Description: Dense vegetation. Grass visible under fallen leaves.



Photograph 5

Location: Slope Toe near Section A'A'
Viewing: Northwest
Description: Slope is well vegetated, no toe active toe erosion is visible, and bedrock is visible on creek bed.



Photograph 6

Location: Slope Toe near Section B-B'
Viewing: Southeast
Description: Another view of the slope toe.



Photograph 7

Location: Slope Toe, Section B-B'
Viewing: Southeast
Description: Slope is generally stable, no toe active toe erosion is visible.



Photograph 8

Location: Slope toe near Section C-C'
Viewing: Southeast
Description: Gabion retaining wall visible along the slope crest. The slope toe is densely vegetated.



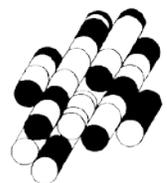
Photograph 9

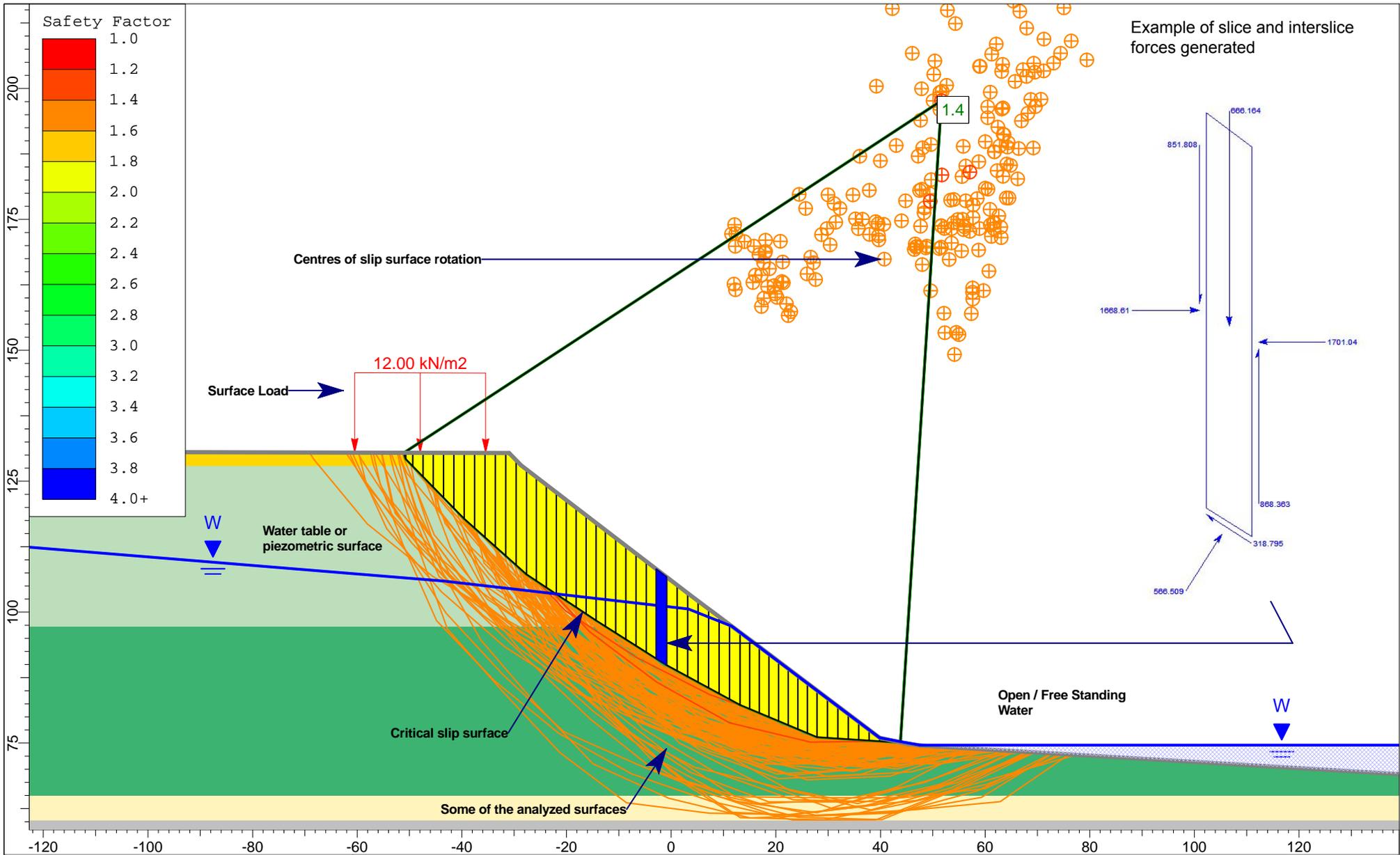
Location: Slope toe near Section D-D'
Viewing: Southeast
Description: The Gabion retaining wall in this area will provide the toe erosion measure as well as grade separation. The wall is relatively fair condition.



APPENDIX D

TERRAPROBE INC.



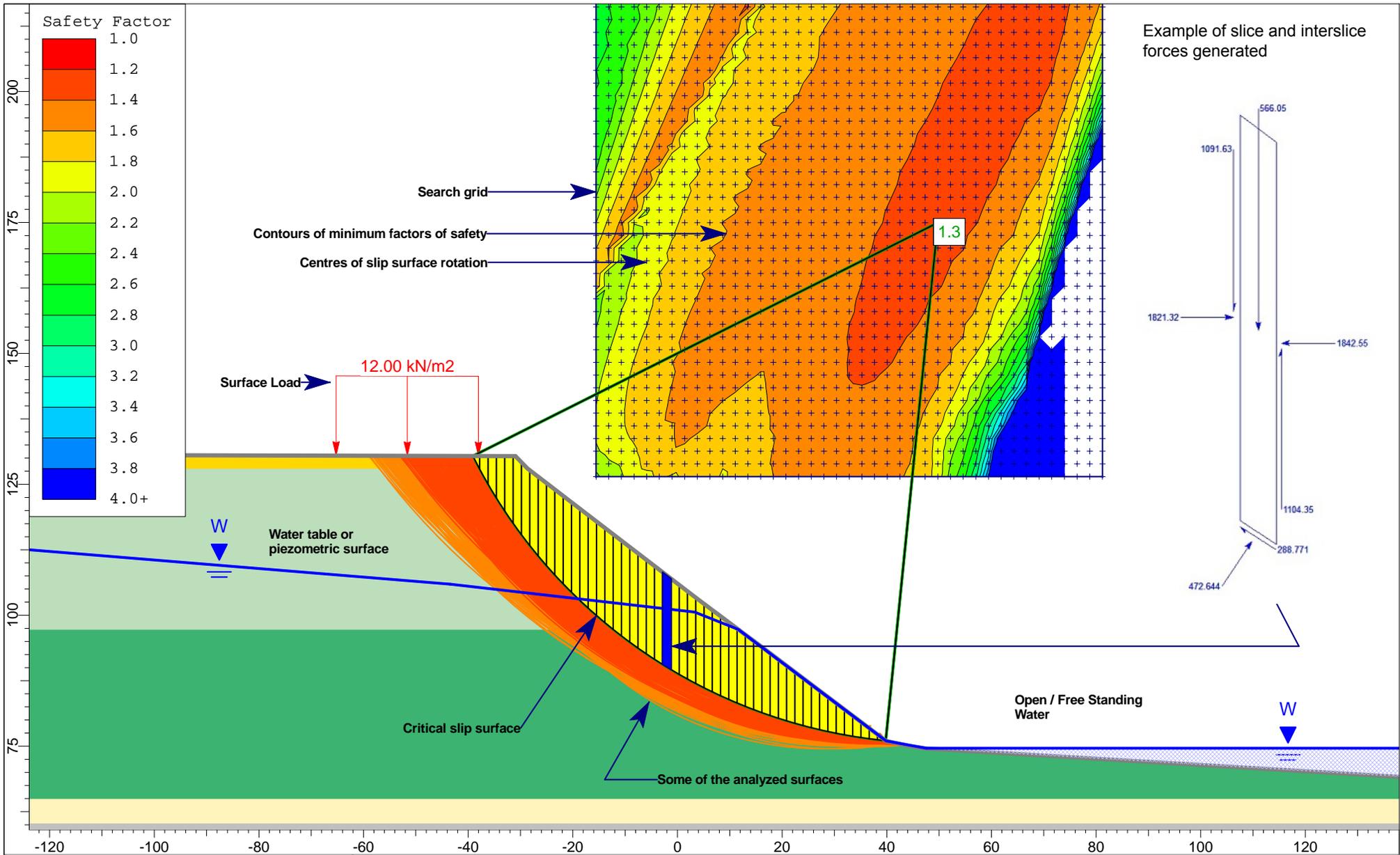


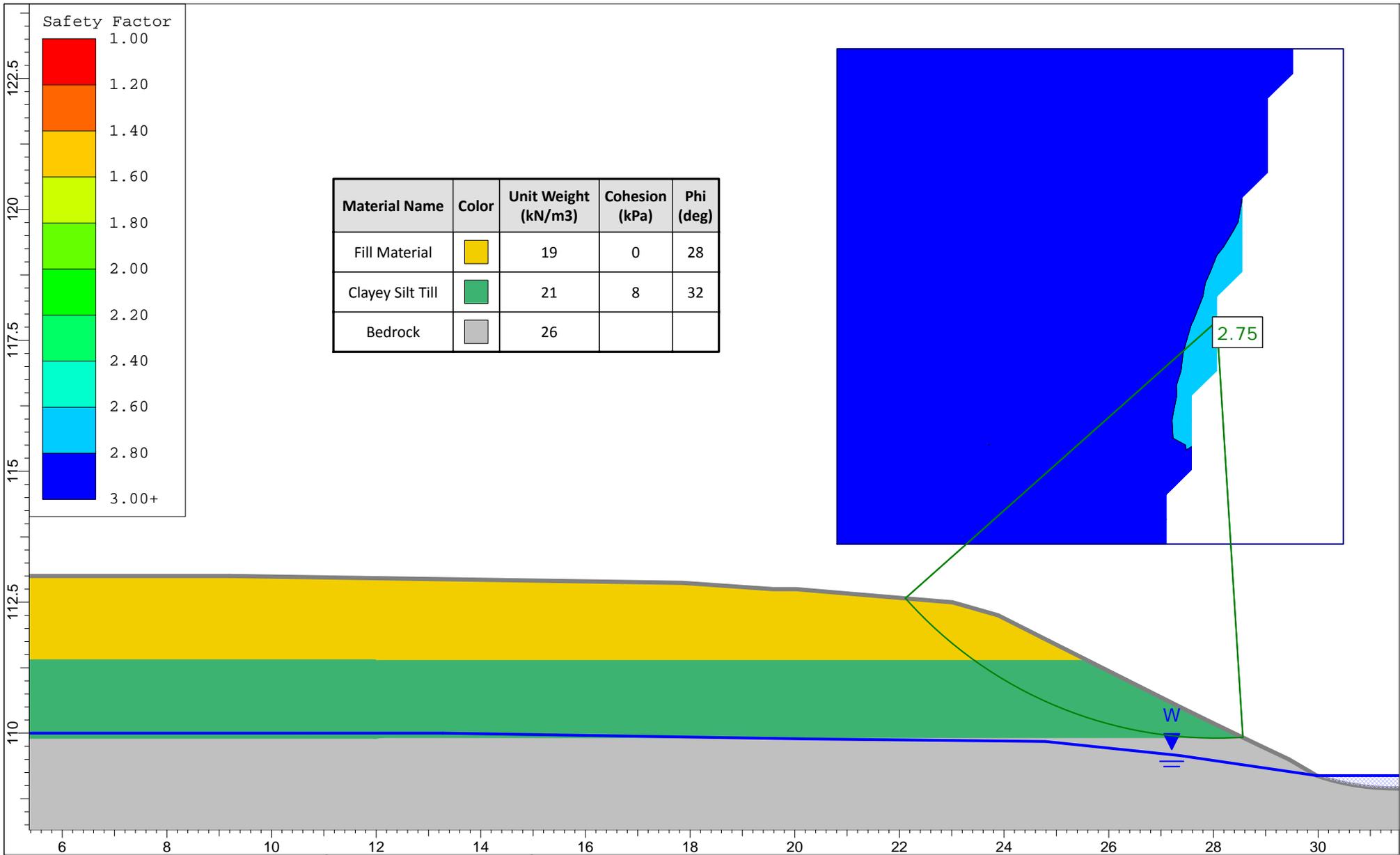
Project

Slope Stability Analysis - Explanation with Slices and Slice Forces

Analysis Description

Non-Circular Analysis (example)





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Construction Materials Inspection & Testing

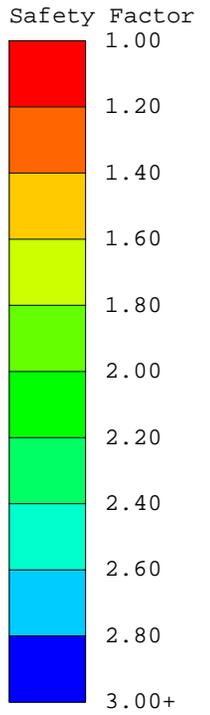
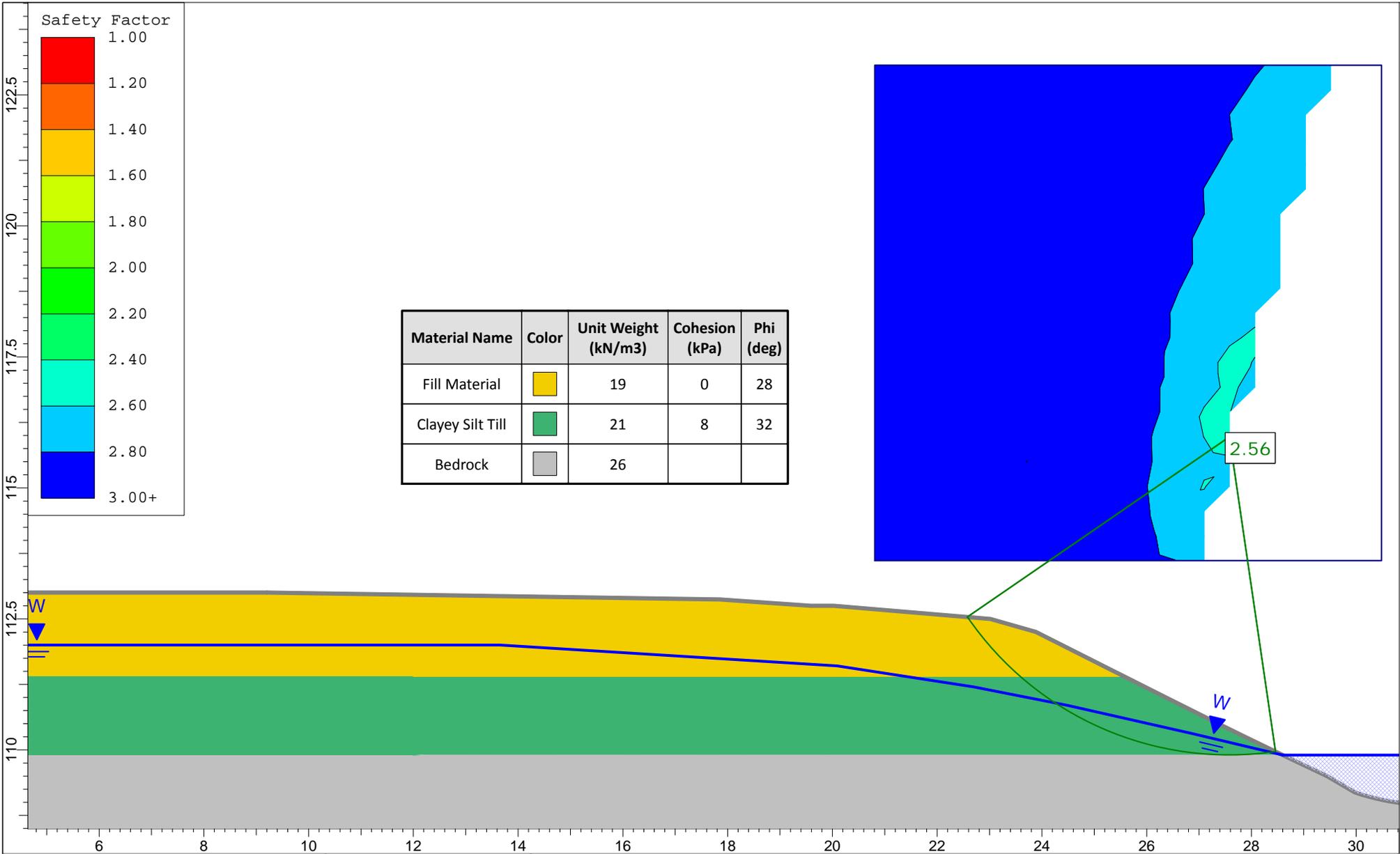
Notes
Refer to appended Slope Stability Analysis Explanation sheets for legend. Refer to cross-sections for inclinations and other pertinent slope information.

Project **1-19-0719-01 2570-2590 Argyle Road, Mississauga**

Analysis **Slope Stability: Section A-A', Existing Conditions, NWL**

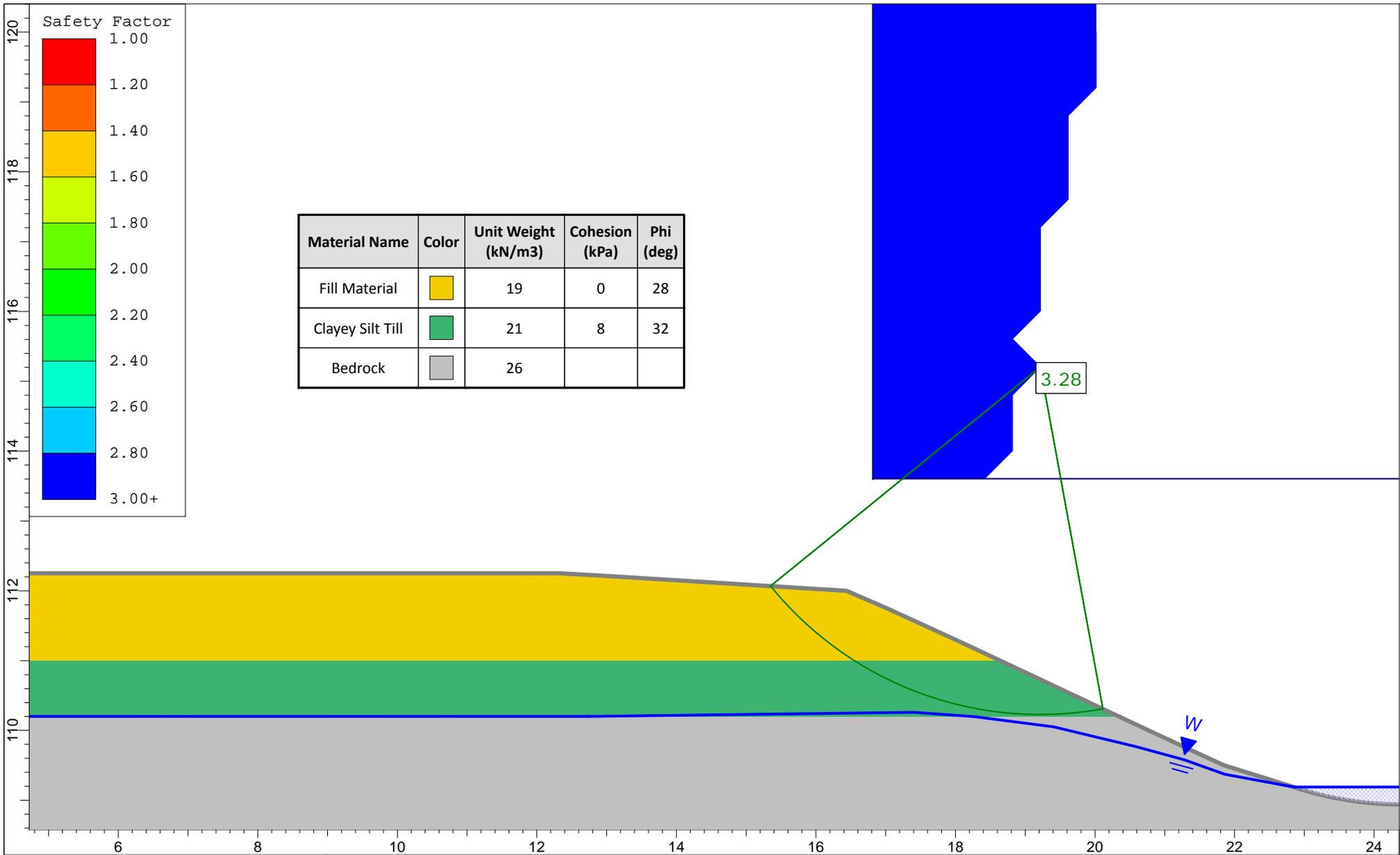
Date 5/21/2020 Scale 1:100 File 2570-2590 Argyle Road.slm

By BV Ref. Topographic Survey from Speight, Van Nostrand & Gibson Ltd.



Material Name	Color	Unit Weight (kN/m ³)	Cohesion (kPa)	Phi (deg)
Fill Material	Yellow	19	0	28
Clayey Silt Till	Green	21	8	32
Bedrock	Grey	26		

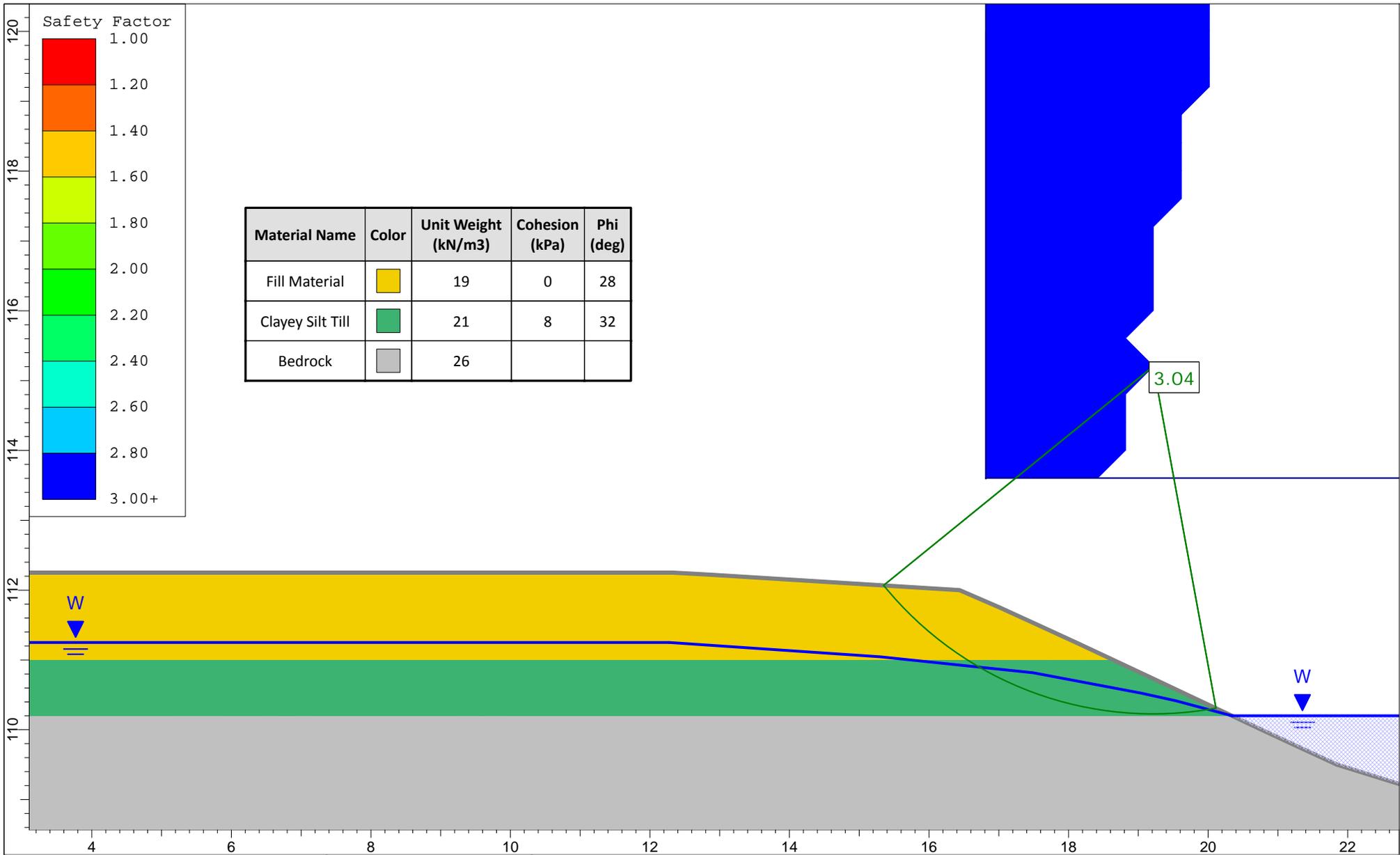
 <p>Terraprobe Consulting Geotechnical & Environmental Engineering Construction Materials Inspection & Testing</p>	Notes	Project	1-19-0719-01 2570-2590 Argyle Road, Mississauga				
	Refer to appended Slope Stability Analysis Explanation sheets for legend. Refer to cross-sections for inclinations and other pertinent slope information.	Analysis	Slope Stability: Section A-A', Existing Conditions, HWL				
		Date	5/21/2020	Scale	1:100	File	2570-2590 Argyle Road.slm
		By	BV	Ref.	Topographic Survey from Speight, Van Nostrand & Gibson Ltd.		




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Notes
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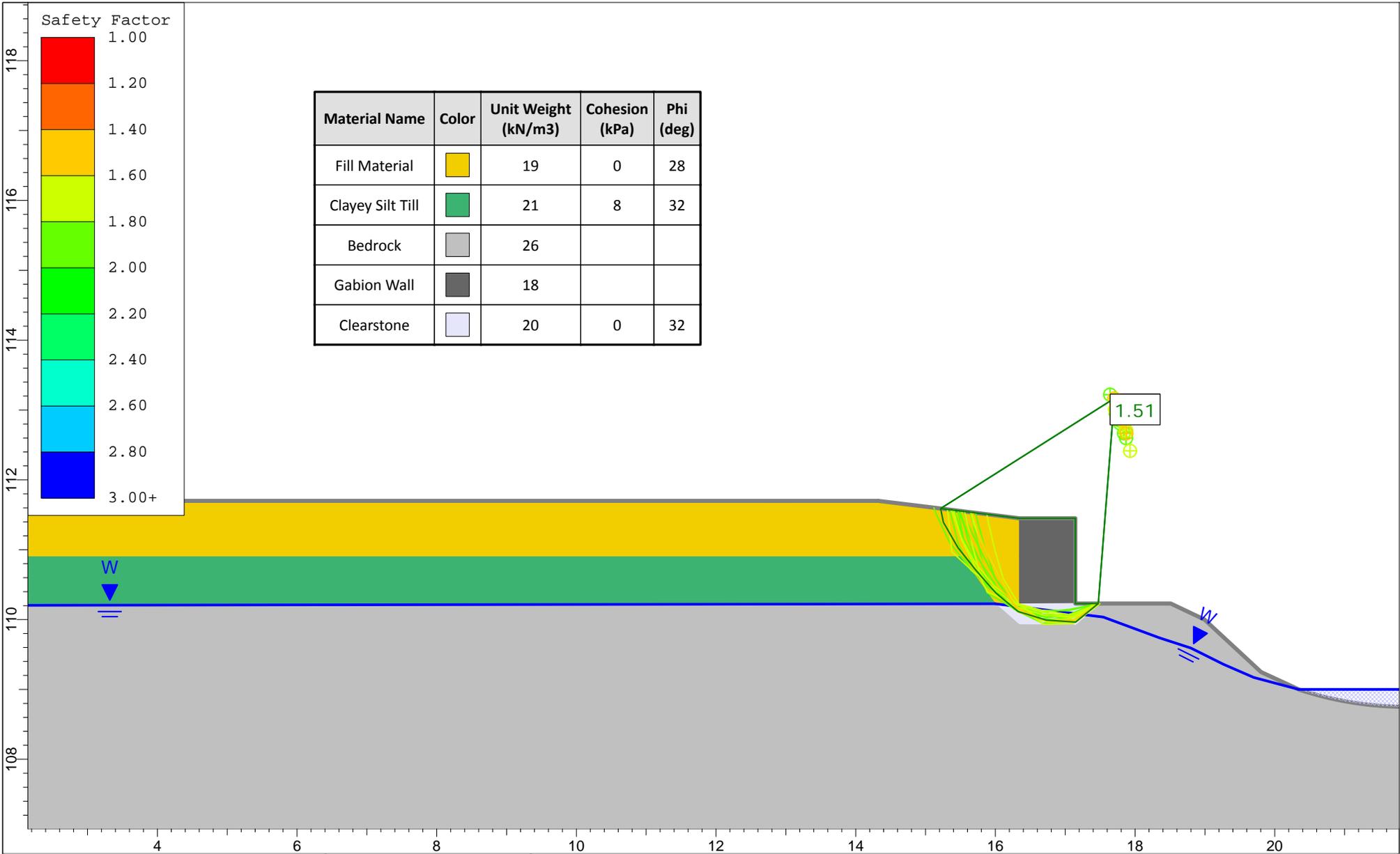
Project 1-19-0719-01 2570-2590 Argyle Road		
Analysis Slope Stability: Section B-B', Existing Conditions, NWL		
Date 5/21/2020	Scale 1:75	File 2570-2590 Argyle Road.slm
By BV	Ref. Topographic Survey from Speight, Van Nostrand & Gibson Ltd.	



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Notes
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Project 1-19-0719-01 2570-2590 Argyle Road		
Analysis Slope Stability: Section B-B', Existing Conditions, HWL		
Date 5/21/2020	Scale 1:75	File 2570-2590 Argyle Road.slm
By BV	Ref. Topographic Survey from Speight, Van Nostrand & Gibson Ltd.	



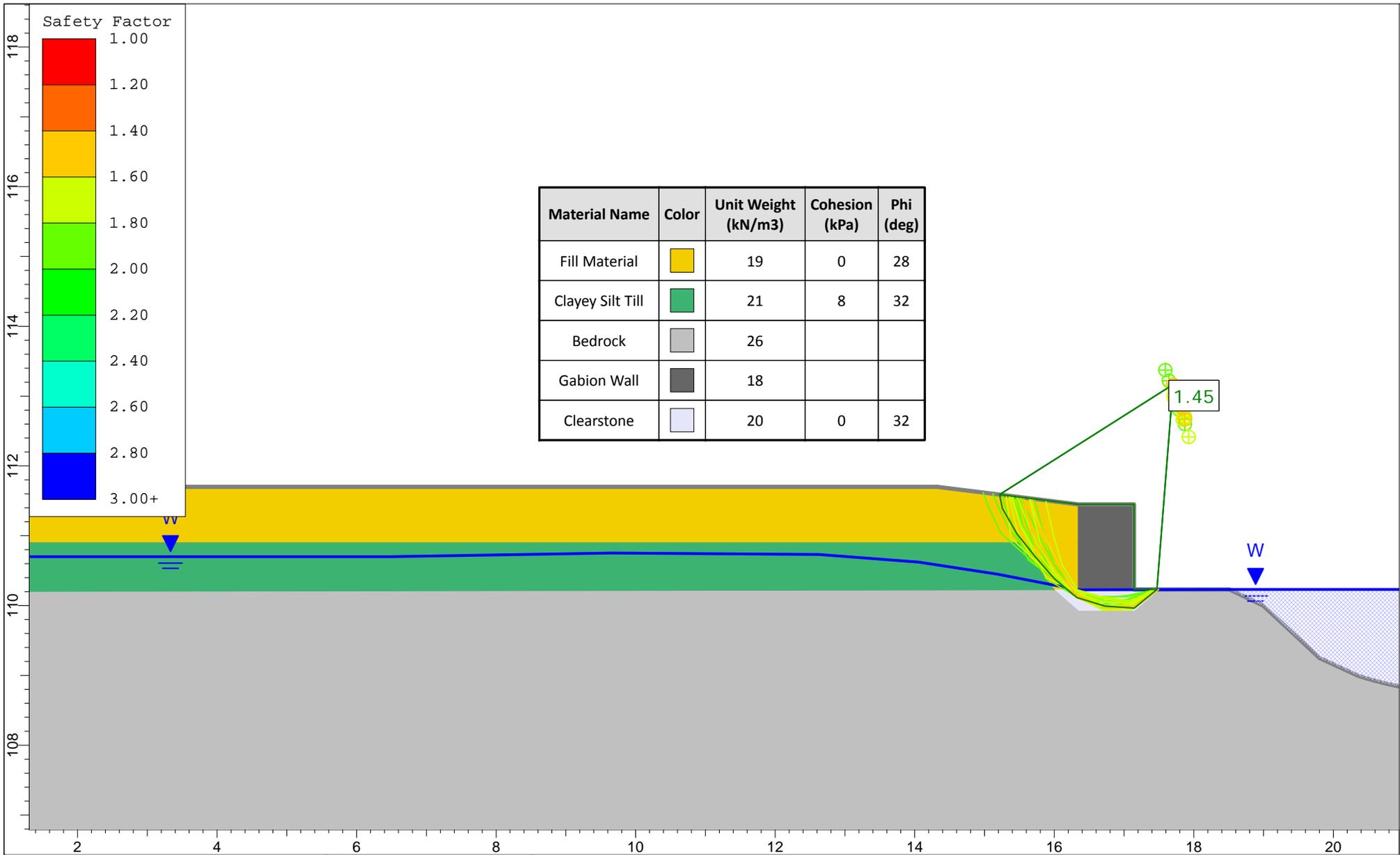
Material Name	Color	Unit Weight (kN/m ³)	Cohesion (kPa)	Phi (deg)
Fill Material	Yellow	19	0	28
Clayey Silt Till	Green	21	8	32
Bedrock	Grey	26		
Gabion Wall	Dark Grey	18		
Clearstone	Light Blue	20	0	32



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Notes
 Refer to appended Slope Stability Analysis Explanation sheets for legend. Refer to cross-sections for inclinations and other pertinent slope information.

Project			1-19-0719-01 2570-2590 Argyle Road		
Analysis			Slope Stability: Section C-C', Existing Conditions, NWL		
Date	5/21/2020	Scale	1:75	File	2570-2590 Argyle Road.slm
By	BV	Ref.	Topographic Survey from Speight, Van Nostrand & Gibson, Ltd.		

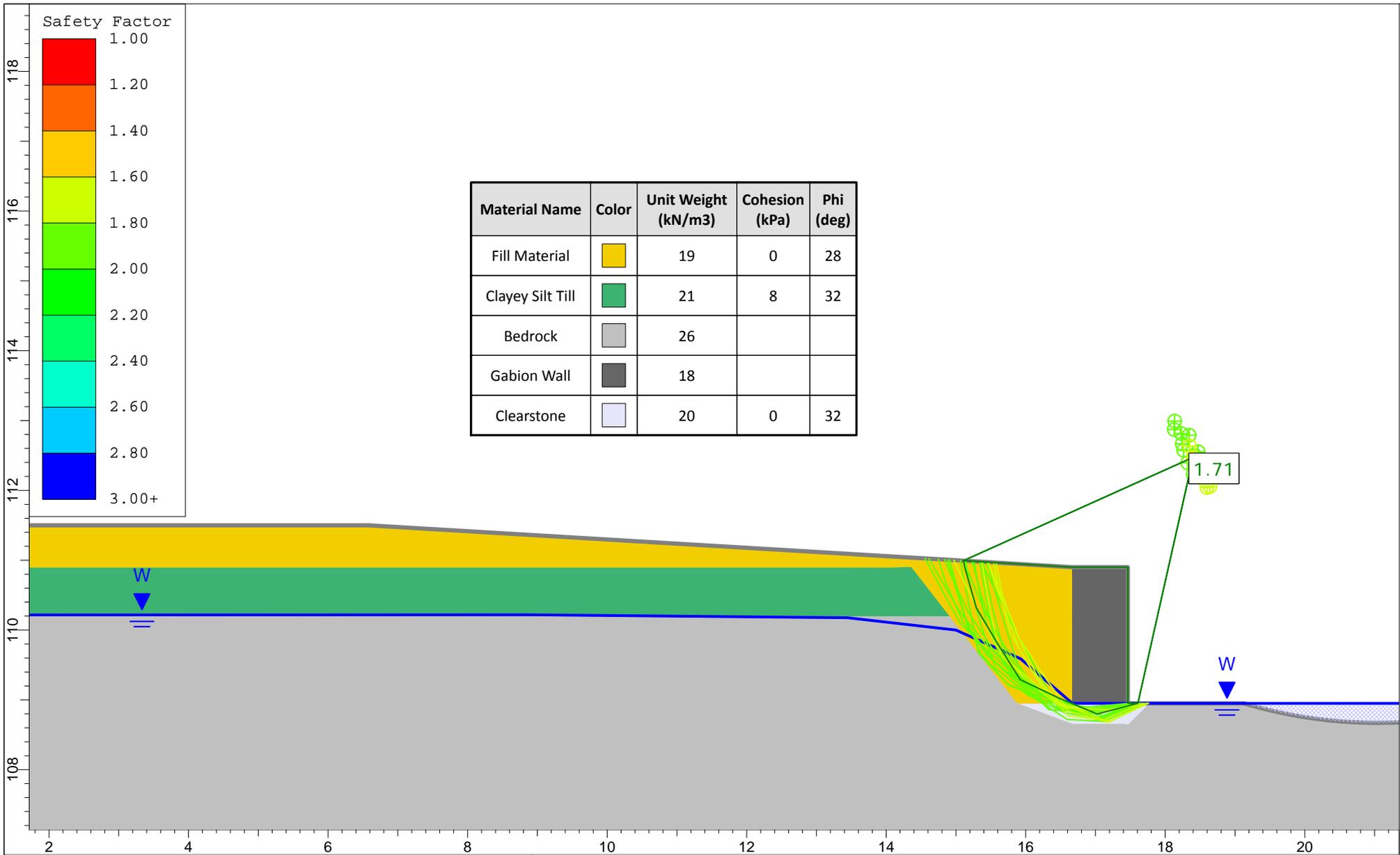


Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Fill Material		19	0	28
Clayey Silt Till		21	8	32
Bedrock		26		
Gabion Wall		18		
Clearstone		20	0	32

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Notes
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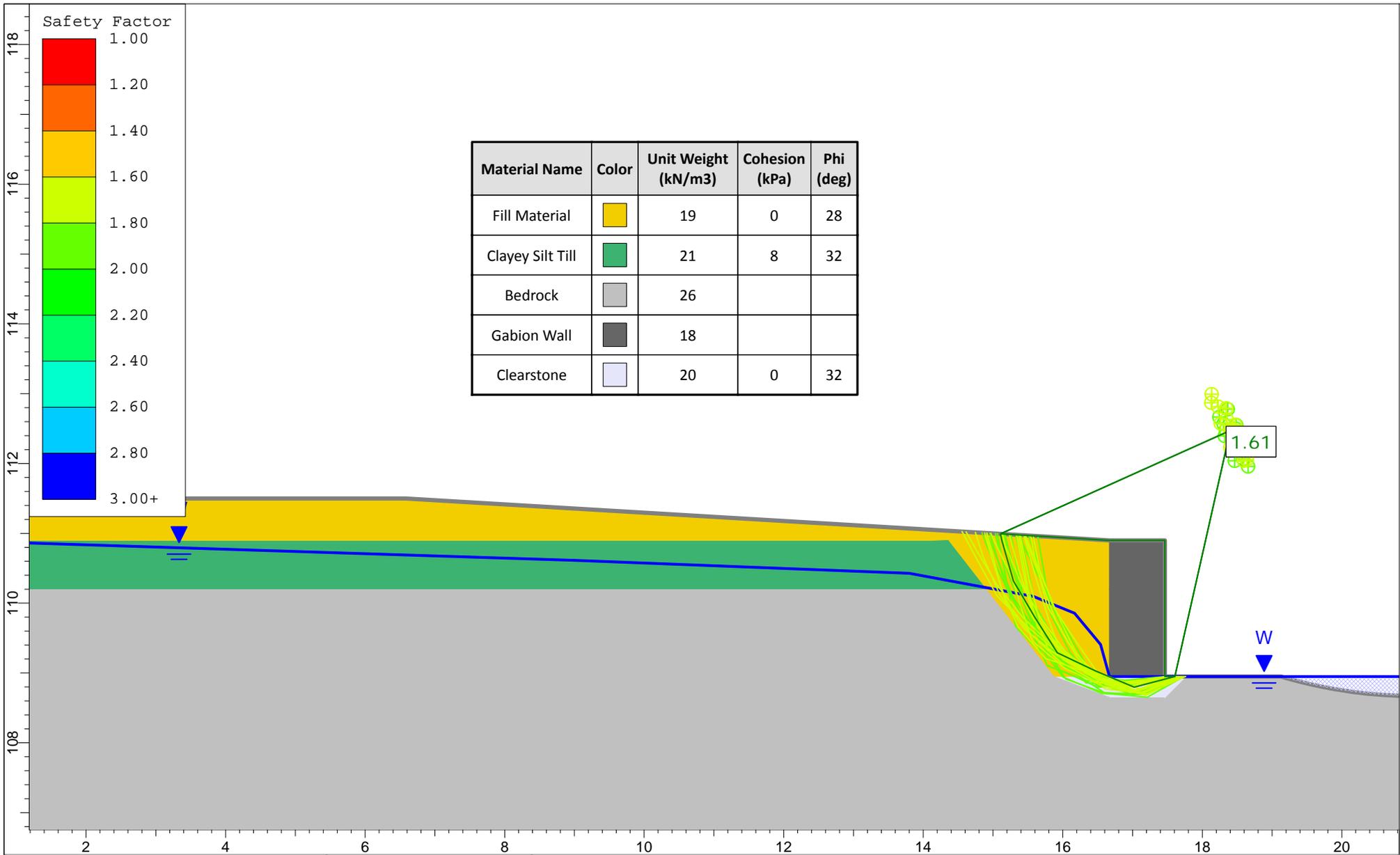
Project 1-19-0719-01 2570-2590 Argyle Road		
Analysis Slope Stability: Section C-C', Existing Conditions, HWL		
Date 5/21/2020	Scale 1:75	File 2570-2590 Argyle Road.slm
By BV	Ref. Topographic Survey from Speight, Van Nostrand & Gibson, Ltd.	



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Notes
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Project 1-19-0719-01 2570-2590 Argyle Road		
Analysis Slope Stability: Section D-D', Existing Conditions, NWL		
Date 5/21/2020	Scale 1:75	File 2570-2590 Argyle Road.scmd
By BV	Ref. Topographic Survey from Speight	

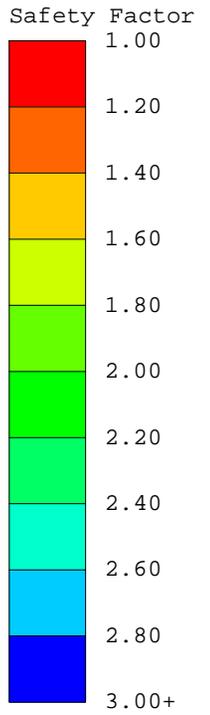
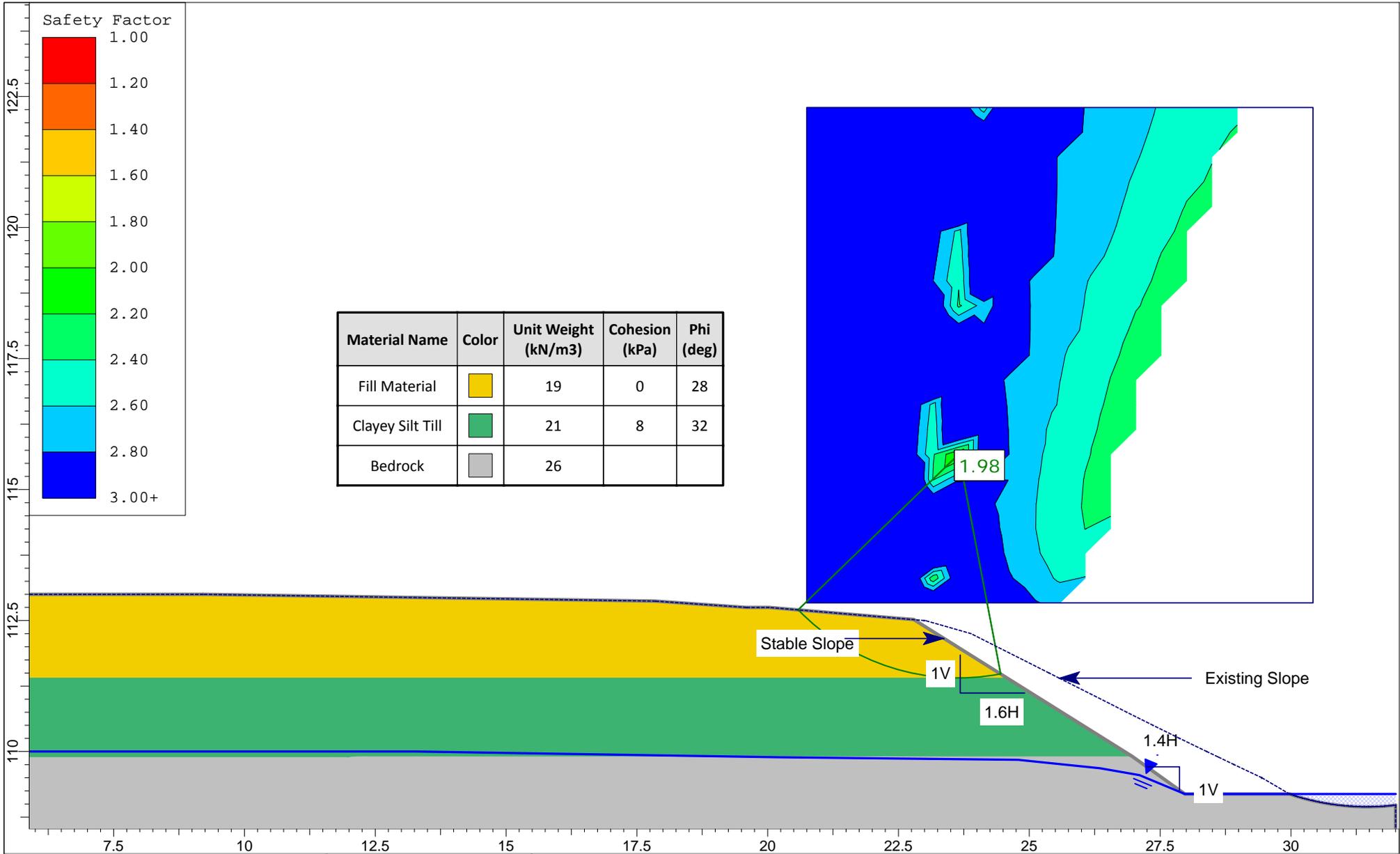


Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Fill Material		19	0	28
Clayey Silt Till		21	8	32
Bedrock		26		
Gabion Wall		18		
Clearstone		20	0	32

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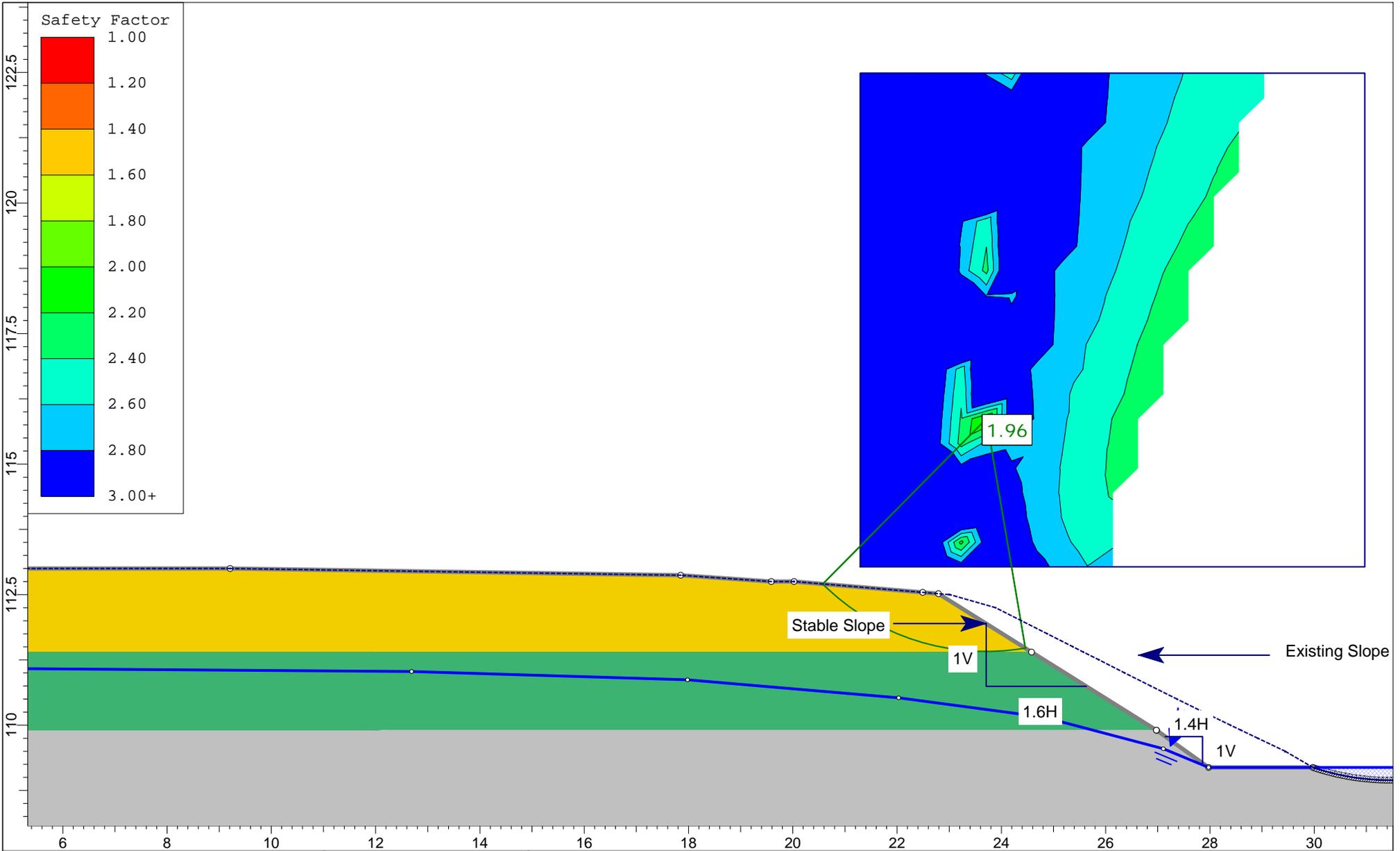
Notes
 Refer to appended Slope Stability Analysis Explanation sheets for legend. Refer to cross-sections for inclinations and other pertinent slope information.

Project 1-19-0719-01 2570-2590 Argyle Road		
Analysis Slope Stability: Section D-D', Existing Conditions, HWL		
Date 5/21/2020	Scale 1:75	File 2570-2590 Argyle Road.slm
By BV	Ref. Topographic Survey from Speight	



Material Name	Color	Unit Weight (kN/m ³)	Cohesion (kPa)	Phi (deg)
Fill Material	Yellow	19	0	28
Clayey Silt Till	Green	21	8	32
Bedrock	Grey	26		

 <p>Terraprobe Consulting Geotechnical & Environmental Engineering Construction Materials Inspection & Testing</p>	<p>Notes Refer to appended Slope Stability Analysis Explanation sheets for legend. Refer to cross-sections for inclinations and other pertinent slope information.</p>	<p>Project 1-19-0719-01 2570-2590 Argyle Road, Mississauga</p>			
		<p>Analysis Slope Stability: Section A-A', with Setback, NWL</p>			
		<p>Date 5/21/2020</p>	<p>Scale 1:100</p>	<p>File 2570-2590 Argyle Road.slm</p>	
		<p>By BV</p>	<p>Ref. Topographic Survey from Speight, Van Nostrand & Gibson Ltd.</p>		



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Notes
 Refer to appended Slope Stability Analysis Explanation sheets for legend. Refer to cross-sections for inclinations and other pertinent slope information.

Project			1-19-0719-01 2570-2590 Argyle Road, Mississauga		
Analysis			Slope Stability: Section A-A', with Setback, HWL		
Date	5/21/2020	Scale	1:100	File	2570-2590 Argyle Road.slm
By	BV	Ref.	Topographic Survey from Speight, Van Nostrand & Gibson Ltd.		