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 the City of Mississauga, Ontario. For the purposes of the description in this report, Argyle Road is considered to align in a 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) twelve-storey 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 structure will be a fourteen-storey tower, plus mechanical penthouse (stepping down to six-storey), and rest on a one-level underground parking structure (P1) with a P1 finished floor elevation of Elev. 109.2 ±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.

- Argyle, Project No. 120325, Progress Set, dated January 24, 2022, by IBI Group.

Terraprobe is also providing a Hydrogeological Study under a separate cover (File No. 1-19-0719-46.1).

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. 1-19-0719-46.1).

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 fourteen-storey building with 4 storeys of above-grade parking, and 1 level of underground parking (P1). The design drawing indicates that the P1 FFE would be set at Elev. 109.2 ±m, approximately 3.8 m below ground floor (113.0 ±m), within the inferred bedrock.

Boreholes 1 to 6 advanced within the proposed building footprint encountered the overburn soil extending to Elev. 109.6 to 110.4 m, which graded into the inferred shale bedrock. Bedrock was not cored and proven at this site, and only inferred based on drilling observations. Therefore, the accurate depth to unweathered/sound bedrock was not determined by our investigation. It is therefore required that Terraprobe inspect and approve the foundation subgrade as unweathered/sound bedrock in order for the following recommendations to be valid.

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

The serviceability limit states (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 on unweathered/sound bedrock. 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:

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

Where:

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 excavated surface should be free of weathered rock and loose fractured stone as well as any standing water. If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided. As per the Ontario Building Code, the foundation excavations must be inspected and approved (by Terraprobe) to ensure the bearing capacities stated below are applicable. If incompetent shale is encountered at the proposed bearing depths during foundation excavation or due to inadequate dewatering, sub-excavation to competent shale subgrade is required under the direction of the geotechnical engineer.

4.2 Earthquake Design Parameters

The Ontario Building Code (OBC) stipulates the methodology for earthquake design analysis. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration and the site classification.

Under Ontario Regulation 88/19, the ministry amended Ontario's Building Code (O. Reg 332/12) to further harmonize Ontario's Building Code with the 2015 National Codes. These changes will help reduce red tape for businesses and remove barriers to interprovincial trade throughout the country. The amendments are based on code change proposals the ministry consulted in 2016 and 2017. The majority of the amendments came into effect on January 1, 2020, which includes structural sufficiency of buildings to withstand external forces and improve resilience.

Seismic hazard is defined in the Ontario Building Code by uniform hazard spectra (UHS) at spectral coordinates of 0.2s, 0.5s, 1.0s and 2.0s and a probability of exceedance of 2% in 50 years. The OBC method uses a site classification system defined by the average soil/bedrock properties (e.g., shear wave velocity (vs), Standard Penetration Test (SPT) resistance, and undrained shear strength (s_u) in the top 30 meters of the site stratigraphy below the foundation level, as set out in the Ontario Building Code. There are 6 site classes from A to F, decreasing in ground stiffness from A, hard rock, to E, soft soil; with site class F used to denote problematic soils (e.g., sites underlain by thick peat deposits and/or liquefiable soils). The site class is then used to obtain peak ground acceleration (PGA), peak ground velocity (PGV) site coefficients F_a and F_v, respectively, used to modify the UHS to account for the effects of site-specific soil conditions.

Based on the above noted information, it is recommended that the site designation for seismic analysis be **Site Class B**, as per the Ontario Building Code. Consideration may be given to conducting a site specific Multichannel Analysis of Surface Waves (MASW) at this site to confirm the average shear wave velocity in the top 30 metres of the site stratigraphy.

The values of the site coefficient for design spectral acceleration at period T, F(T), and of similar coefficients F(PGA) and F(PGV) shall conform to Tables 4.1.8.4.B. to 4.1.8.4.I. using linear interpolation for intermediate values of PGA.

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/m^3)

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:

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

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, (γ - 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:

$$\boldsymbol{P = K[\gamma h + q]}$$

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: $\mathbf{R = N \tan \varphi}$. This is an unfactored resistance. The factored resistance at ULS is $\mathbf{R_f = 0.8\ N \tan \varphi}$.

4.4 Basement Floor Slab Design Parameters

The P1 level slab is to be made near Elev. 109.2 ±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. 1-19-0719-46.1), 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. 1-19-0719-46.1).

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 of Mississauga does not permit shoring system encroachments on the City's property. Shoring and associated works are to be wholly within private lands, including excavation supports such as soldier piles and lagging. Exceptions for tiebacks may be considered under a Shoring Permit Application.

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:

$$\boldsymbol{P} = \boldsymbol{K}[\boldsymbol{\gamma H} + \boldsymbol{q}]$$

Where, $\boldsymbol{P}$ = the horizontal pressure at depth, **H** (kPa)
$\boldsymbol{K}$ = the earth pressure coefficient (see Section 4.3.1)
$\boldsymbol{H}$ = the total depth of the excavation (m)

γ = the bulk unit weight of soil, (kN/m3)

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 2A.

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 2A). 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 2A, 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	Exiting 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 2A 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 statues, 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.

LICENSED PROFESSIONAL ENGINEER
L ZHANG
100108586
April 11, 2022
PROVINCE OF ONTARIO

Blasco Vijayabaskaran, P.Eng.
Geotechnical Engineer

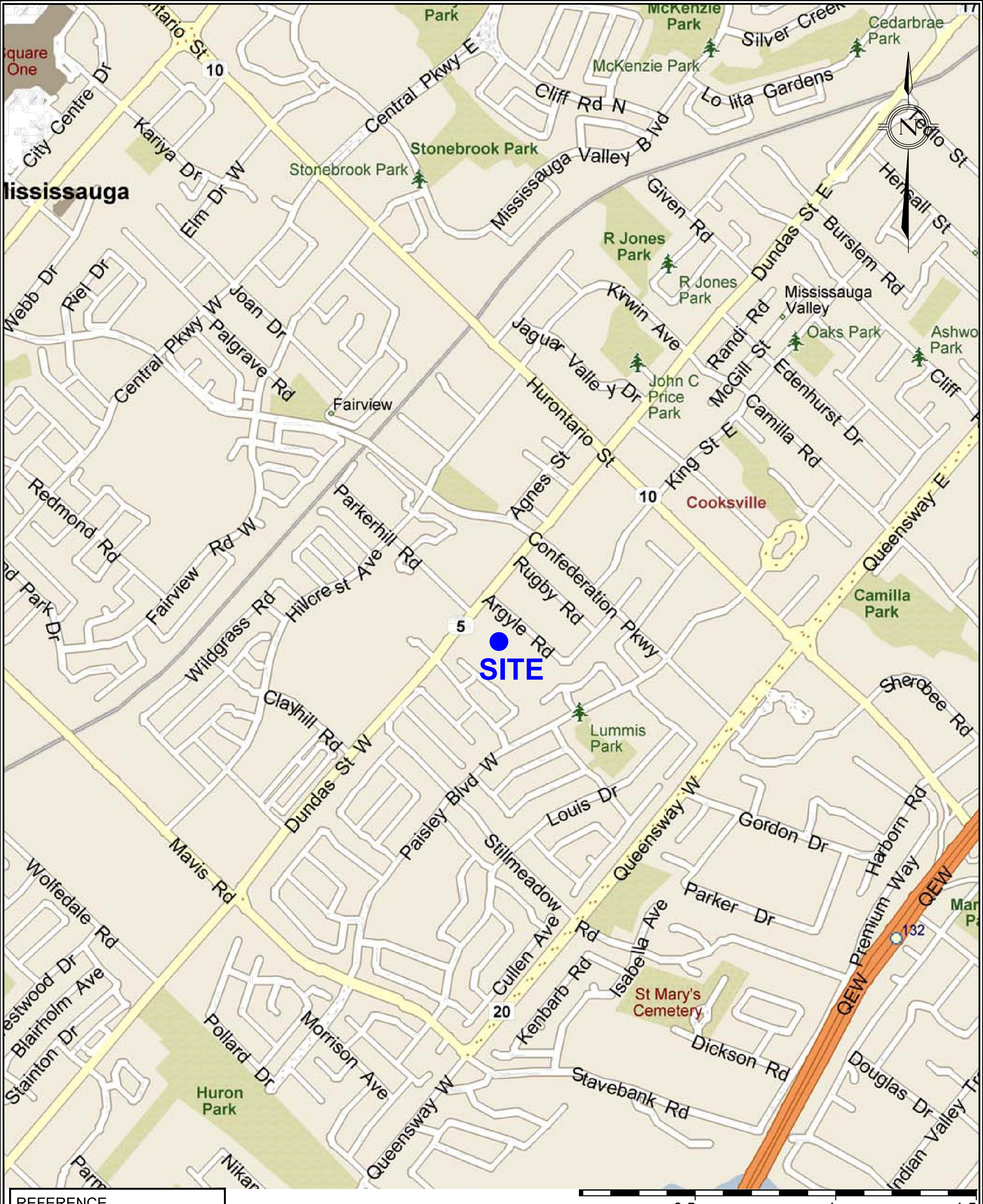
Seth Zhang, M.Eng., M.Sc., P.Eng.
Associate

ENCLOSURES

TERRAPROBE INC.

FIGURES

TERRAPROBE INC.



REFERENCE
Microsoft Streets & Trips 2012

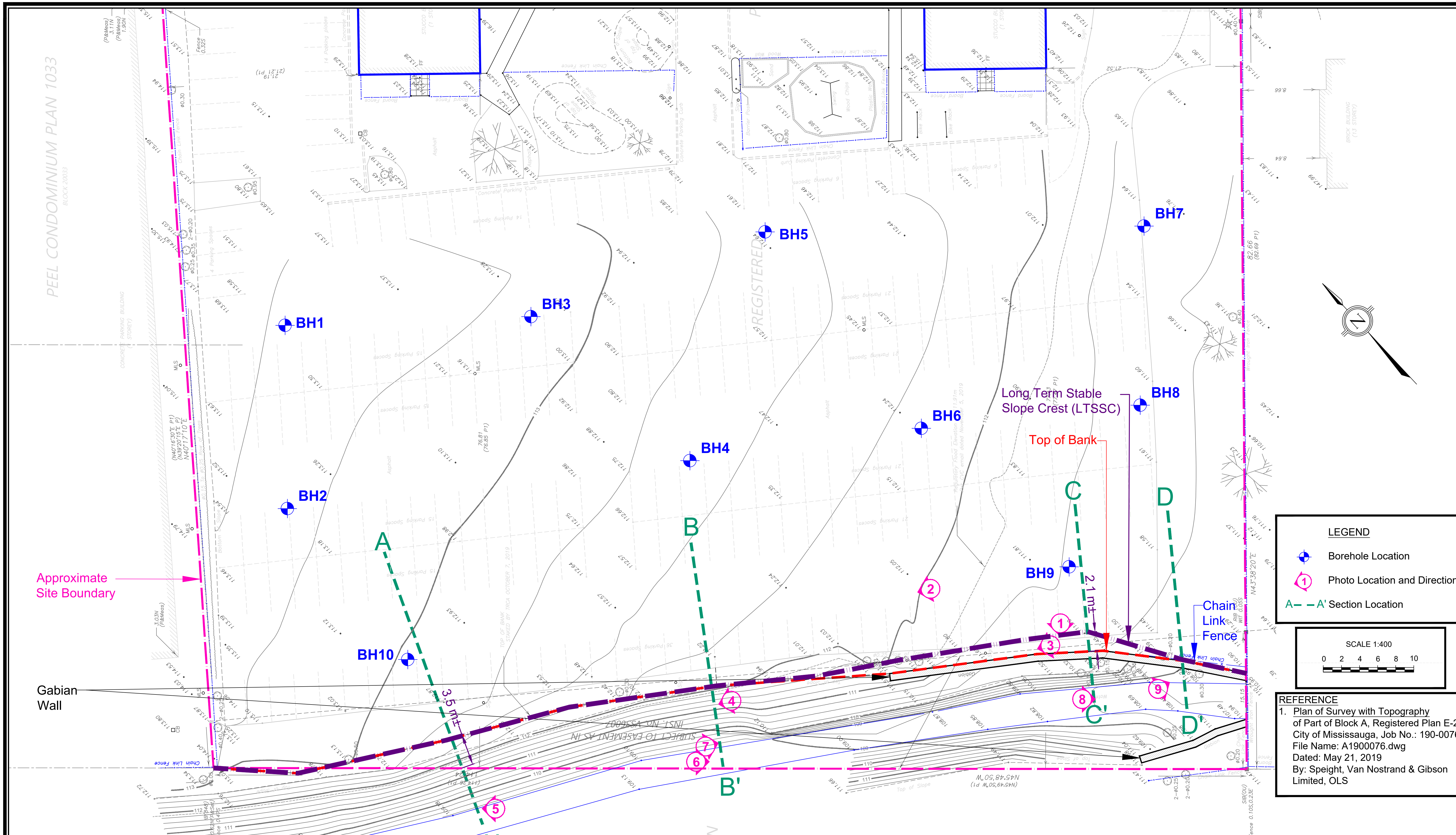
Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title: SITE LOCATION PLAN

File No. 1-19-0719-01

FIGURE : 1

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



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

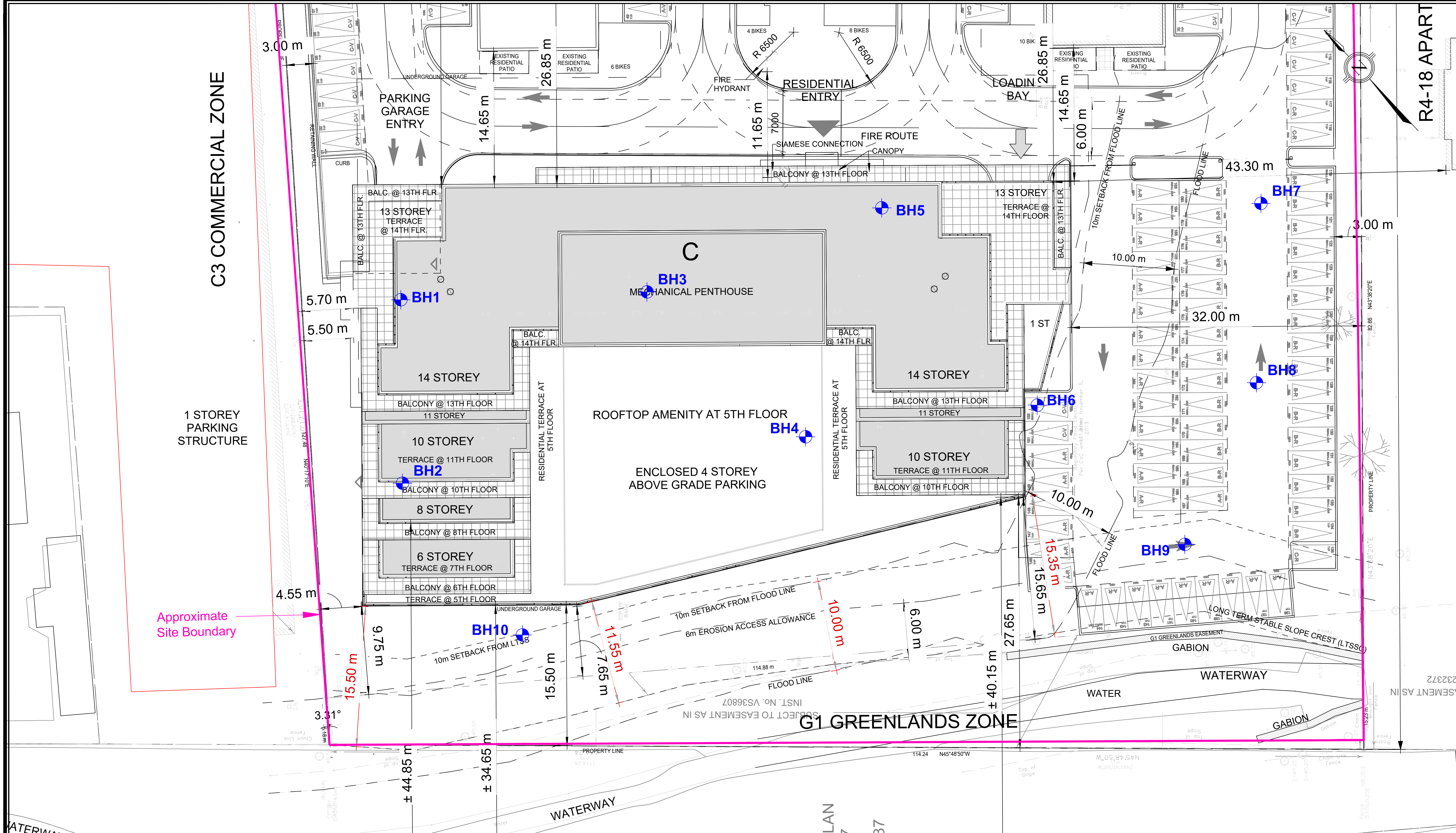
Terraprobe

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

Title: BOREHOLE LOCATION AND SITE FEATURES PLAN EXISTING CONDITIONS

File No. 1-19-0719-01

FIGURE : 2A



REFERENCE
Drawing provided in digital format by IBI Group, drawing file "120325_Argyle- SITE PLAN (2)", received Feb. 03, 2022 by email.

LEGEND
Approximate Borehole Location

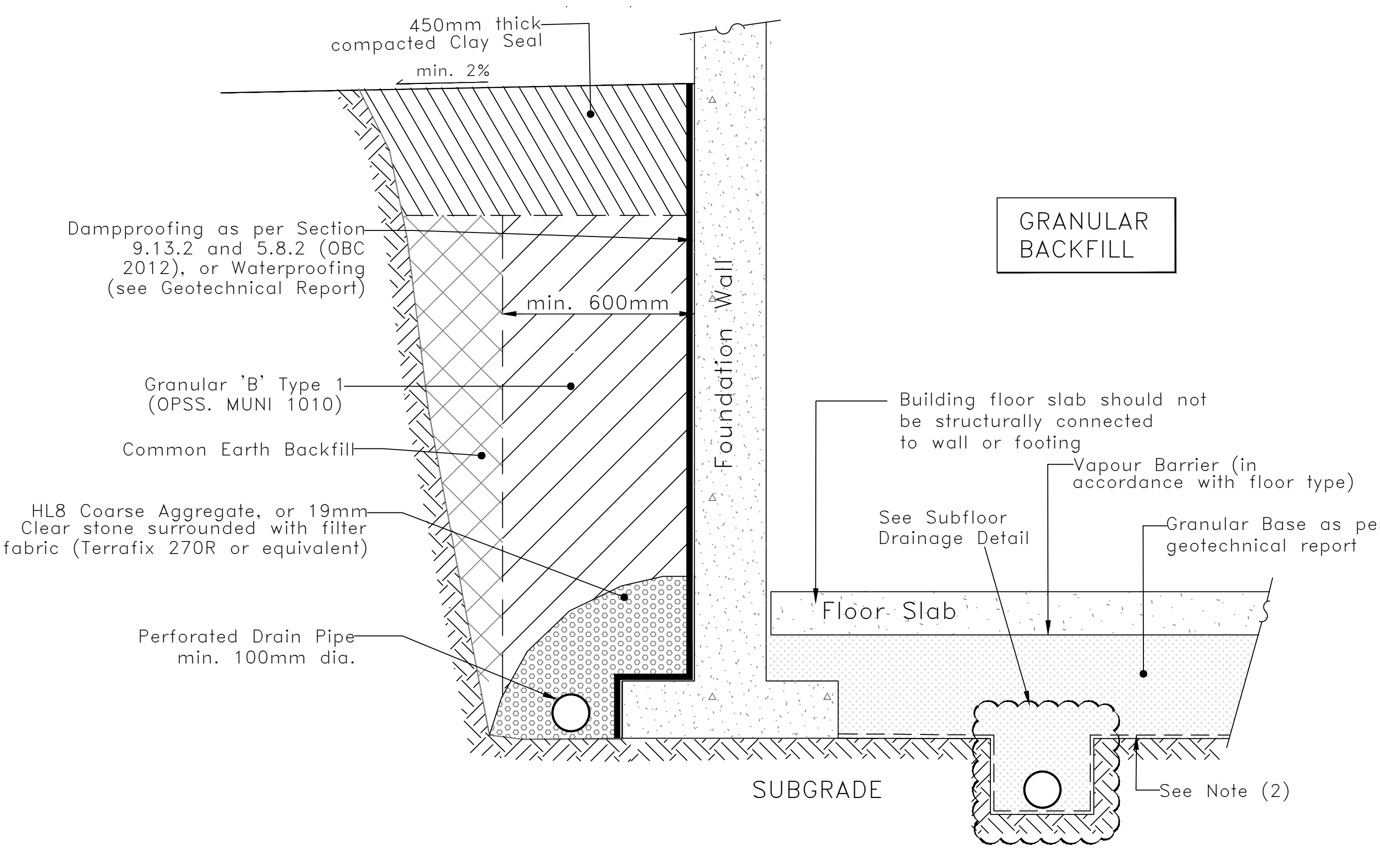
SCALE 1:400
0 2 4 6 8 10m

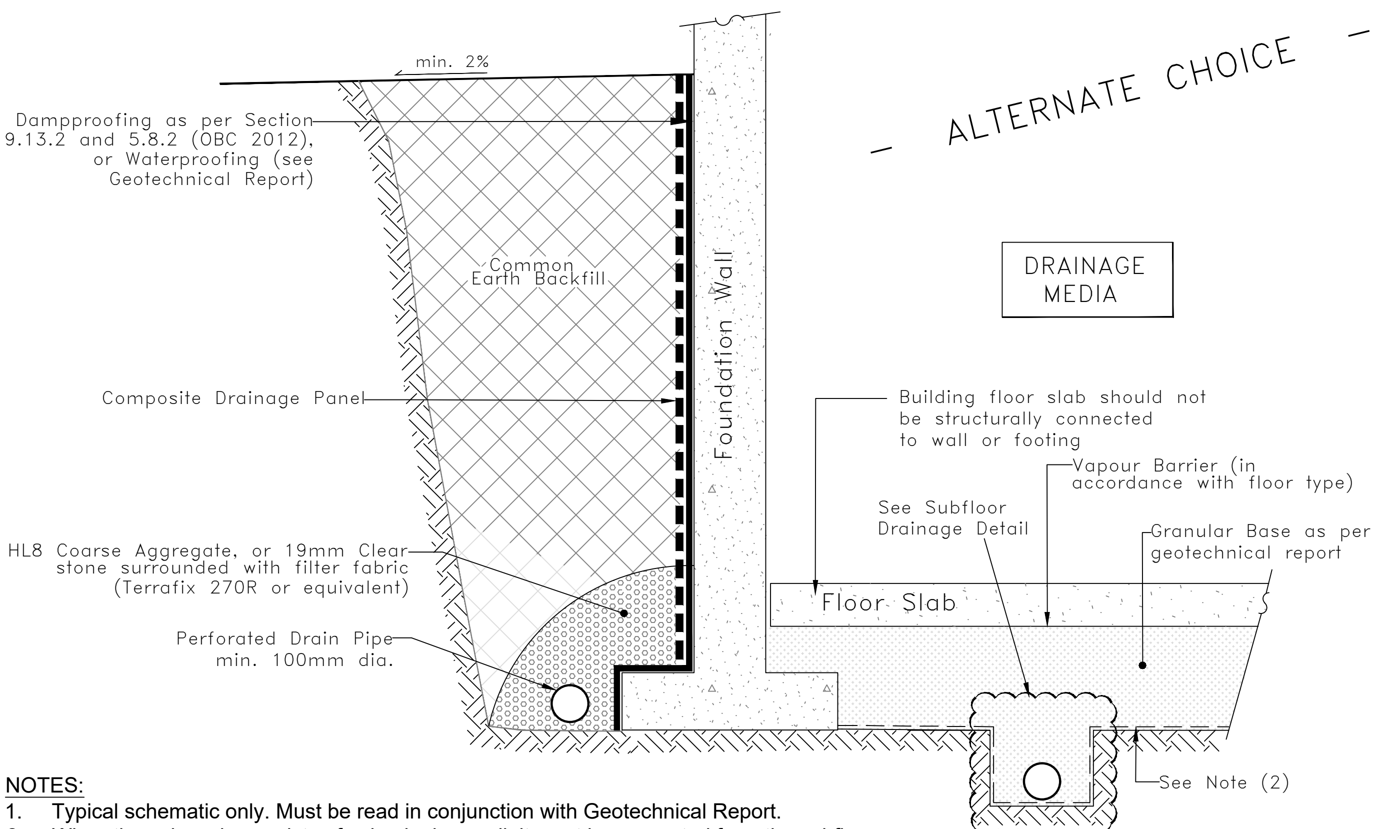
Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:
BOREHOLE LOCATION AND SITE FEATURES PLAN
PROPOSED CONDITION

File No. 1-19-0719-01

FIGURE : 2B





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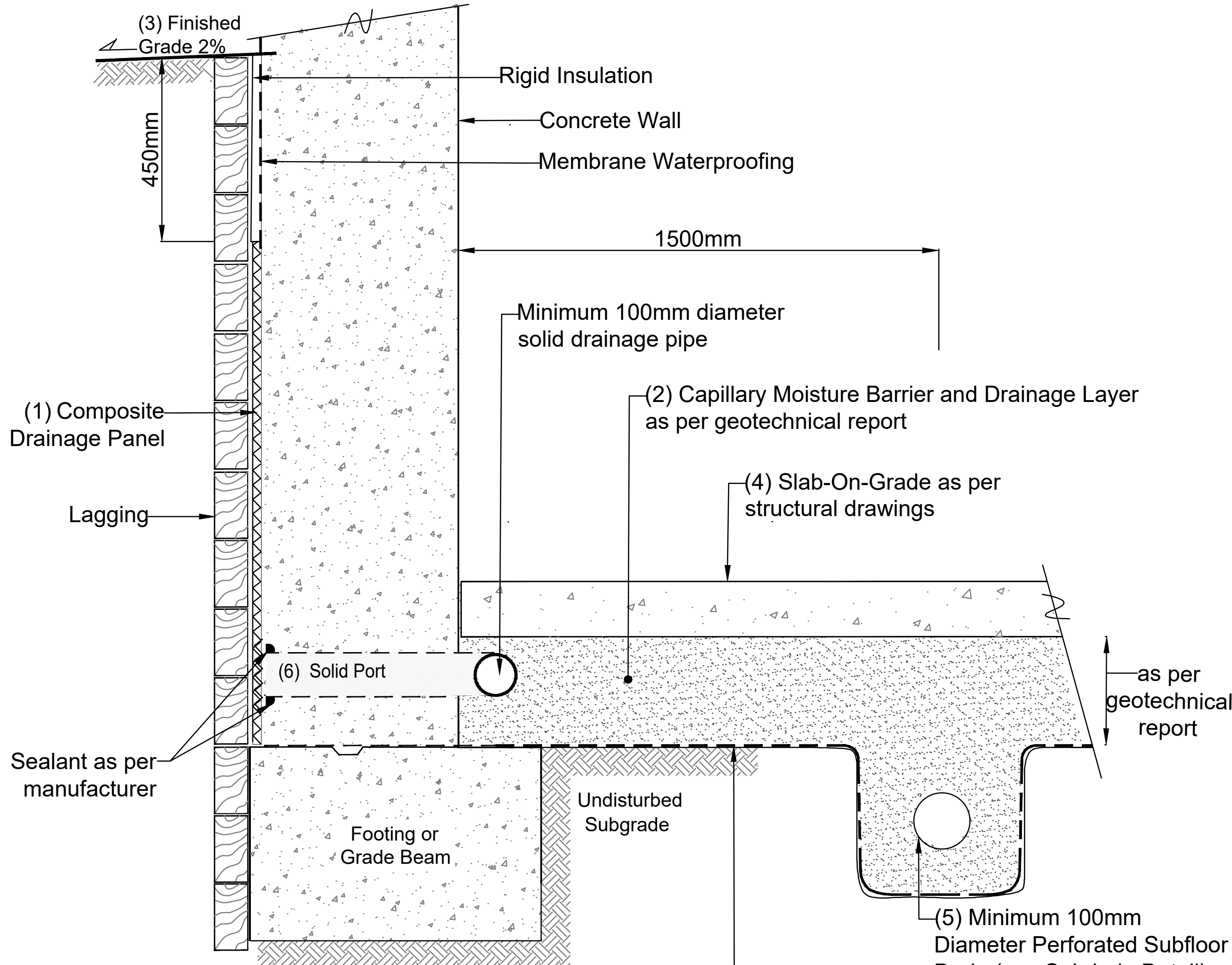
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

TYPICAL BASEMENT 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).

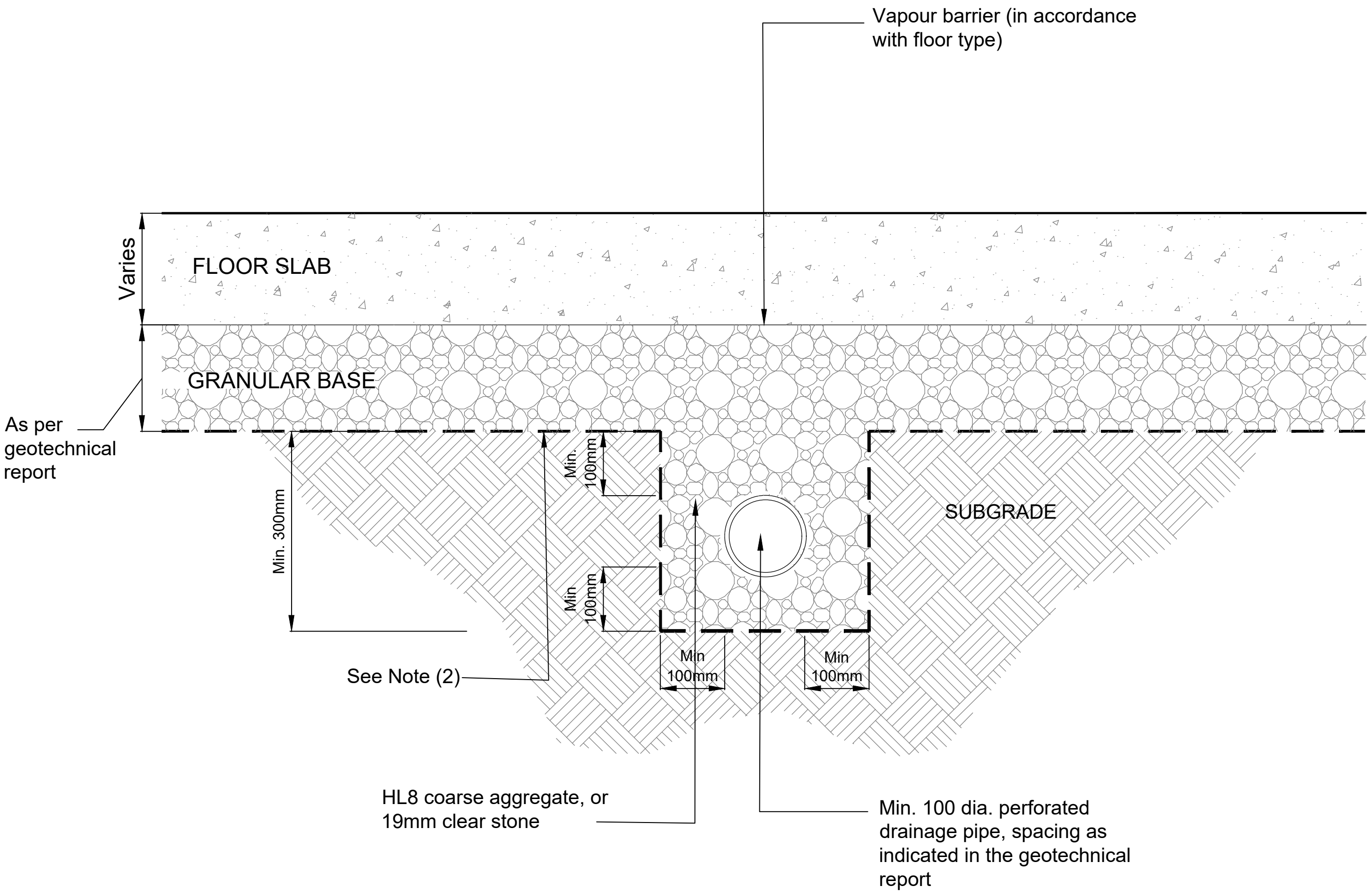
8) Geotechnical report contains specific details. Final detail must be reviewed before system is considered acceptable to use.

N.T.S.

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

Title:

SCHEMATIC DRAINAGE DETAIL
SOLDIER PILE & LAGGING SHORING SYSTEM
(DRAINED BASEMENT CONDITION)



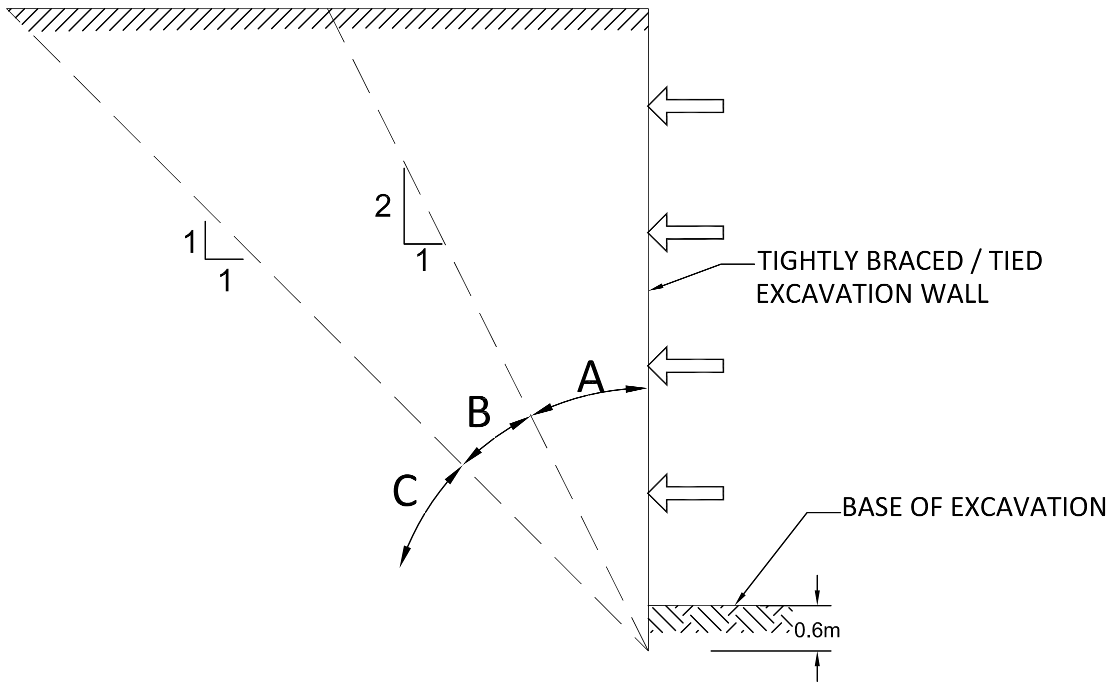
NOTES:

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

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
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:

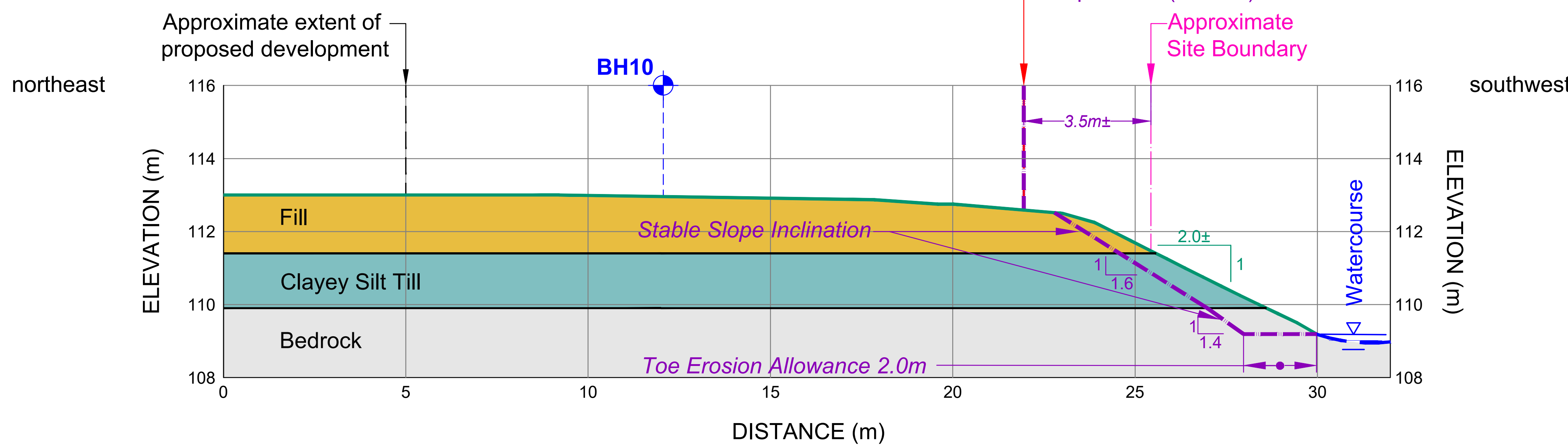
User's Guide - NBC 2005 Structural Commentaries
(Part 4 of Division B) - Commentary K

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

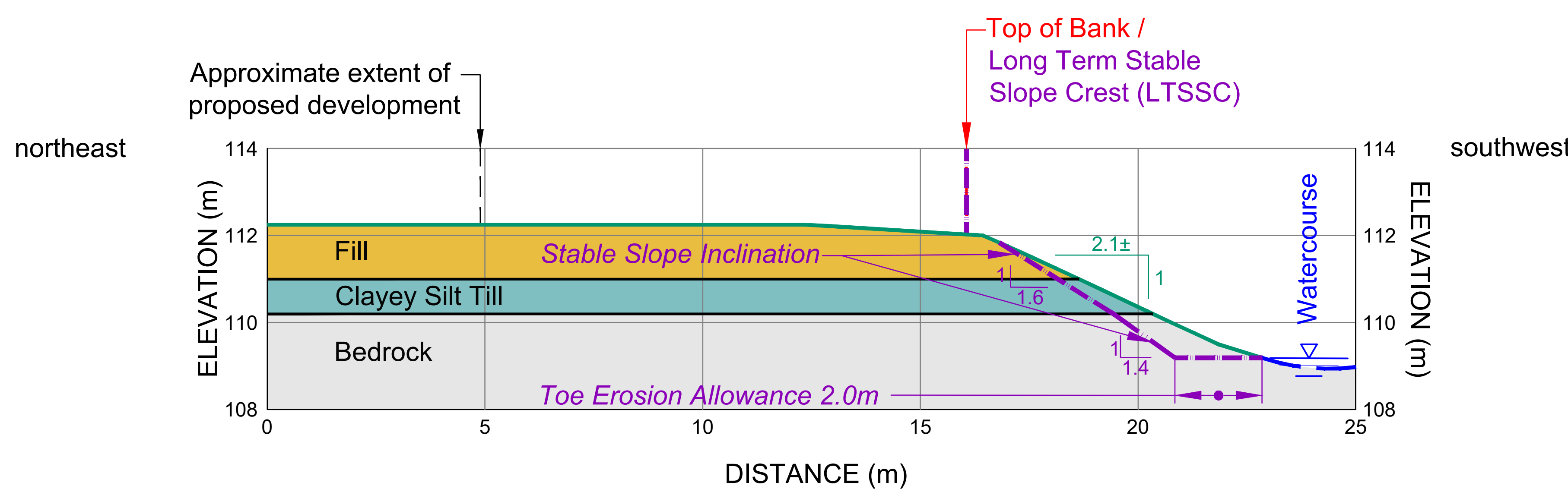
Title:

GUIDELINES FOR UNDERPINNING SOILS

SECTION A-A'



SECTION B-B'



SCALE 1:150

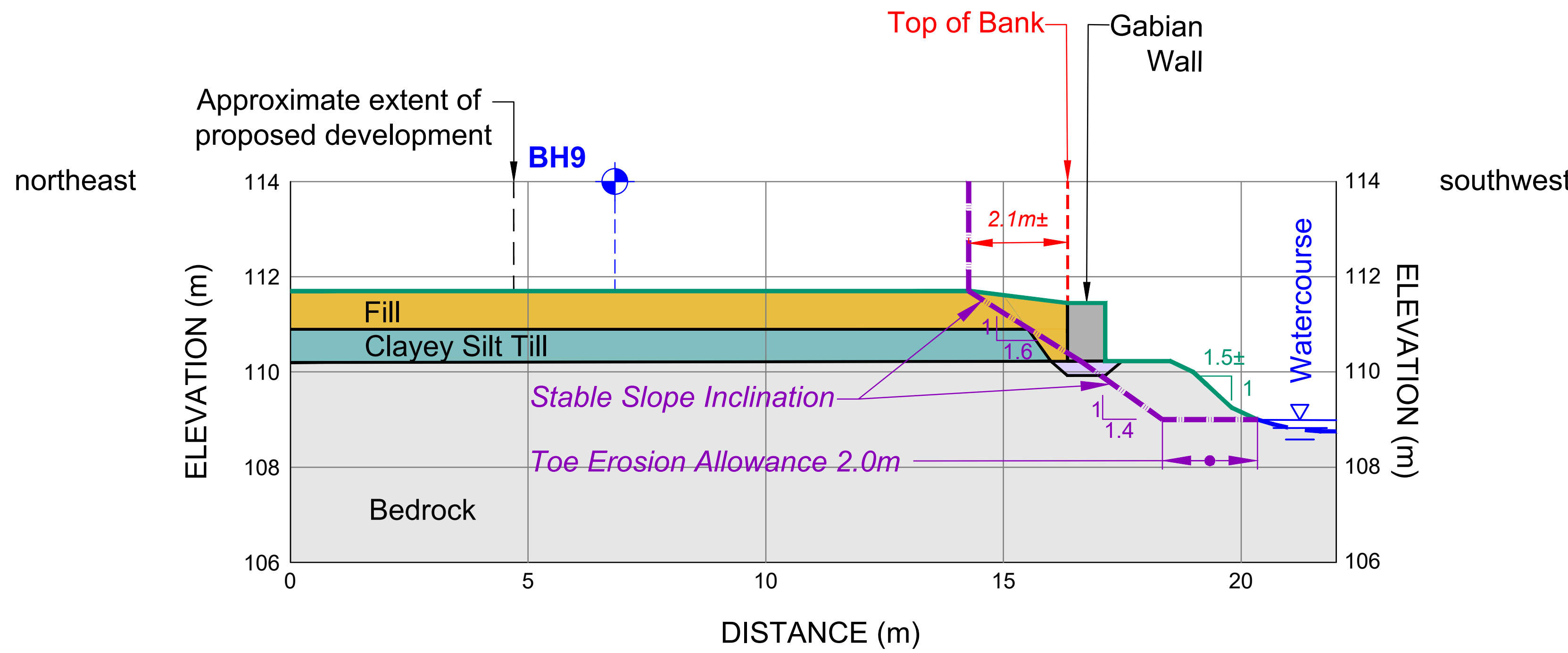
0 1 2 3 4 5

Terraprobe

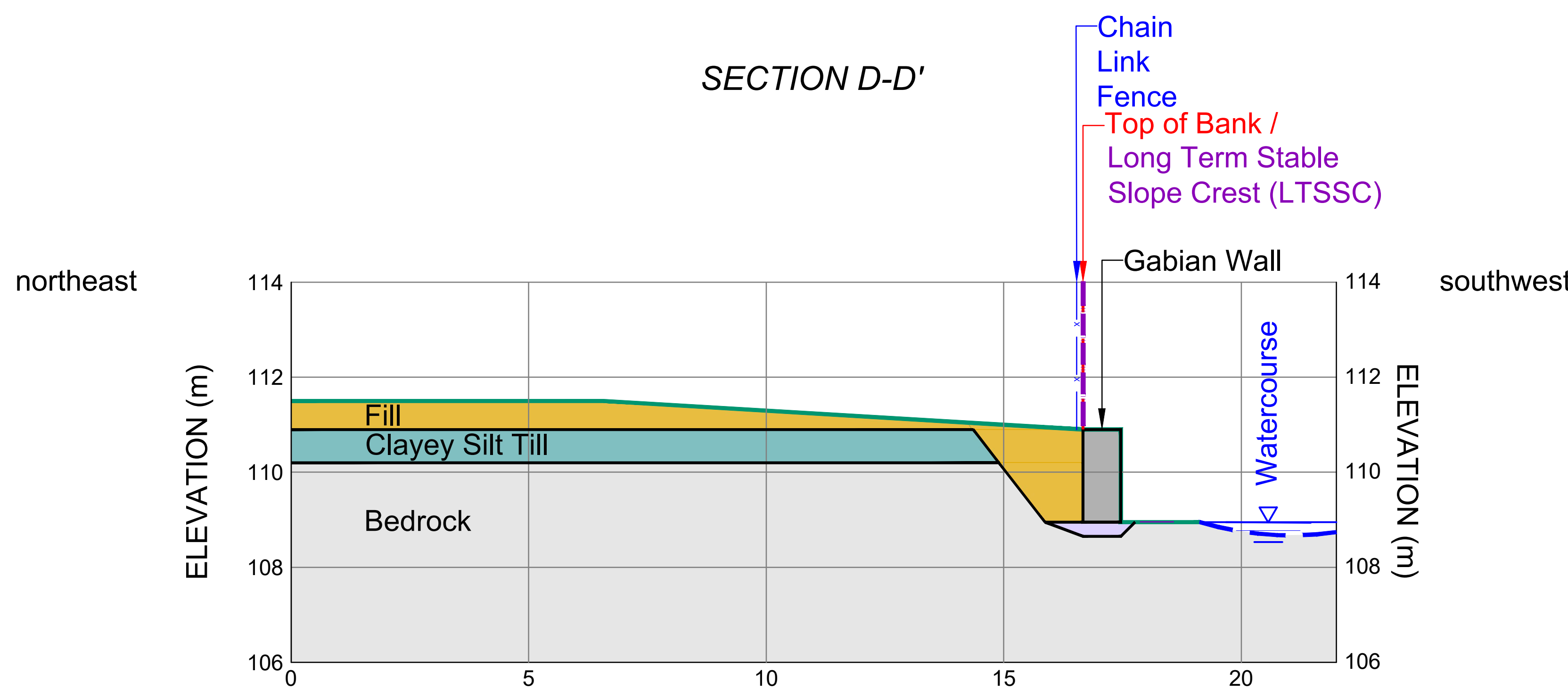
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:	SLOPE CROSS SECTIONS
File No.	1-19-0719-01
FIGURE :	6A

SECTION C-C'



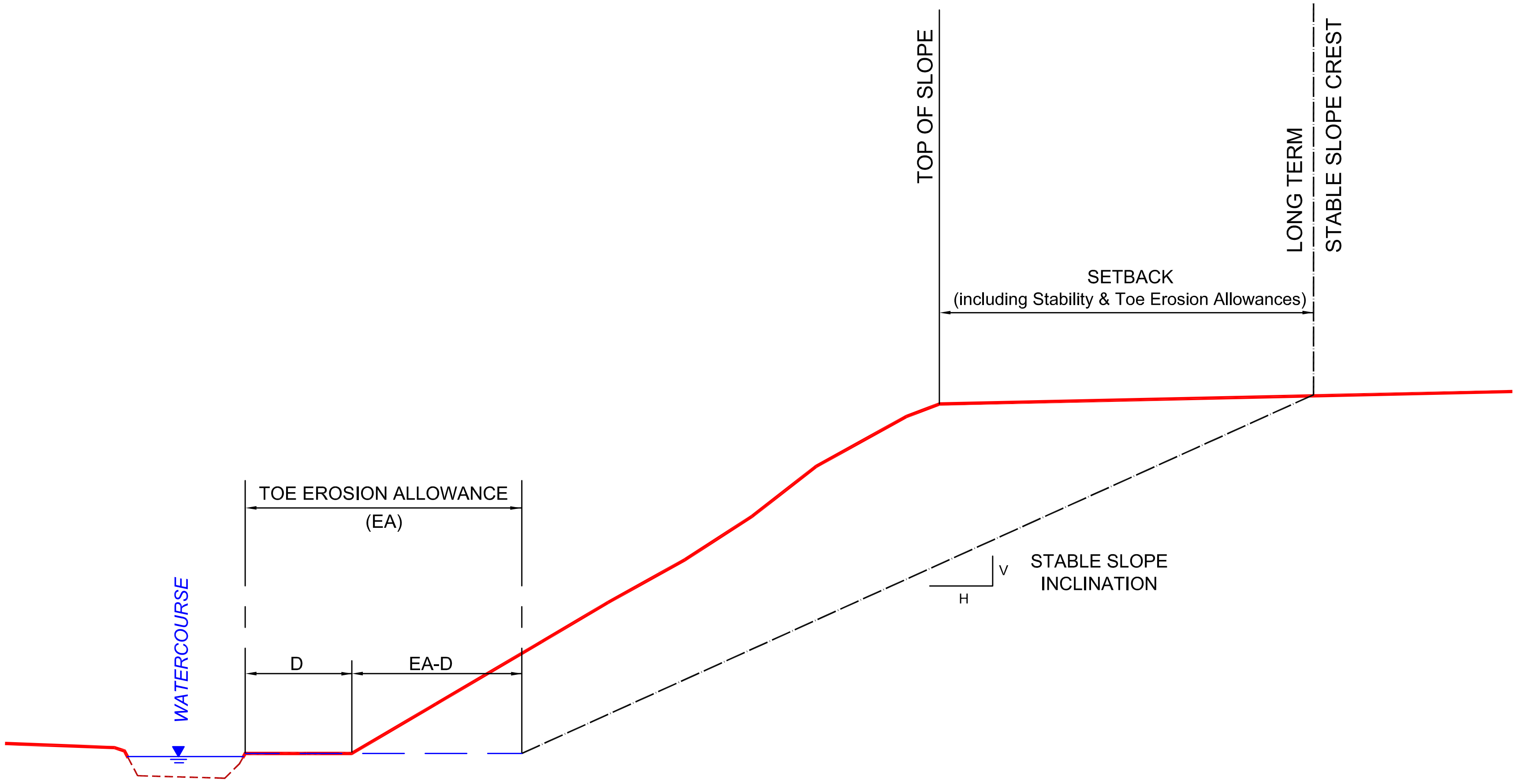
SECTION D-D'



SCALE 1:150
0 1 2 3 4 5

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

Title:	SLOPE CROSS SECTIONS
File No.	1-19-0719-01
FIGURE :	6B



LEGEND

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

EA = Erosion Allowance

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

Title:

LONG TERM STABLE SLOPE CREST MODEL

APPENDIX A

TERRAPROBE INC.

SAMPLING METHODS

AS	auger sample
CORE	cored sample
DP	direct push
FV	field vane
GS	grab sample
SS	split spoon
ST	shelby tube
WS	wash sample

PENETRATION RESISTANCE

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.).

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.)."

COHESIONLESS SOILS

Compactness	'N' value
very loose	< 4
loose	4 – 10
compact	10 – 30
dense	30 – 50
very dense	> 50

COHESIVE SOILS

Consistency	'N' value	Undrained Shear Strength (kPa)
very soft	< 2	< 12
soft	2 – 4	12 – 25
firm	4 – 8	25 – 50
stiff	8 – 15	50 – 100
very stiff	15 – 30	100 – 200
hard	> 30	> 200

COMPOSITION

Term (e.g)	% by weight
trace silt	< 10
some silt	10 – 20
silty	20 – 35
sand *and* silt	> 35

TESTS AND SYMBOLS

MH	mechanical sieve and hydrometer analysis
w, w_c	water content
w_L, LL	liquid limit
w_P, PL	plastic limit
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

	Unstabilized water level
	1st water level measurement
	2nd water level measurement
	Most recent water level measurement
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

LOG OF BH1

Terraprobe

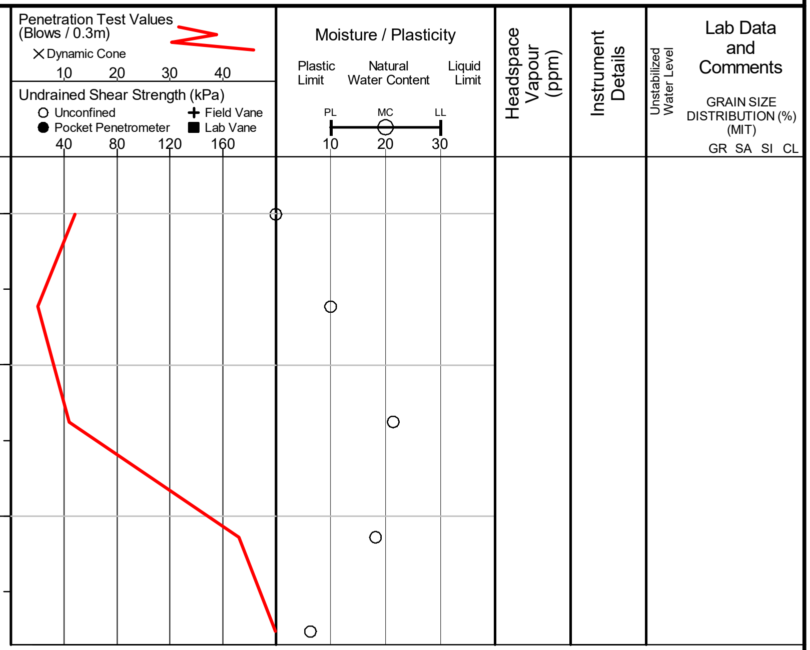
Project No.	: 1-19-0719-01	Client	: Ranee Management	Originated by	: DH
Date started	: December 3, 2019	Project	: 2570 - 2590 Argyle Road	Compiled by	: BV
Sheet No.	: 1 of 1	Location	: Mississauga, Ontario	Checked by	: SZ

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

Rig type : B-37 Drilling Method : Solid stem augers

Depth Scale (m)	Elev Depth (m)	SOIL PROFILE Description	Graphic Log	SAMPLES Number	Type	SPT 'N' Value	Elevation Scale (m)	Moisture Content
0	113.4	**GROUND SURFACE**						
		90mm **ASPHALTIC CONCRETE**						
		FILL, clayey silt, trace sand, trace gravel, with shale fragments, stiff, greyish brown, moist		1	SS	12	113	
1		...at 0.8 m, firm		2	SS	5		
		...at 1.5 m, strong hydrocarbon odour, stiff		3	SS	11	112	
2	111.1 2.3	**CLAYEY SILT**, sandy, trace gravel, hard, brown with orange inclusions, moist (GLACIAL TILL)		4	SS	43	111	
3	110.4	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)		5	SS	50 / 25mm		
	110.2 3.2	**END OF BOREHOLE**						

Borehole was dry and open upon completion of drilling.



file: 1-19-0719 bh logs.gpj

Terraprobe

LOG OF BH2

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

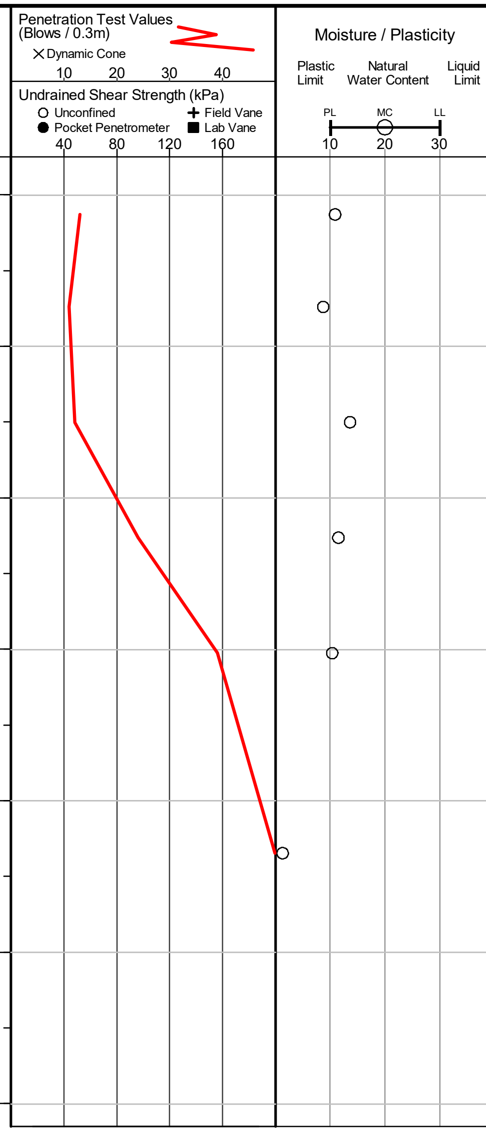
Position : E: 611271, N: 4825498 (UTM 17T)

Elevation Datum : Geodetic

Rig type : B-37

Drilling Method : Solid stem augers

Elev / Depth (m)	Description (SOIL PROFILE)	Sample Number	Type	SPT 'N' Value	Lab Data and Comments
113.3	GROUND SURFACE				
	90mm **ASPHALTIC CONCRETE**				
	FILL, clayey silt, trace sand, trace gravel, trace asphalt pieces, shale fragments, stiff, greyish brown, moist	1	SS	13	
		2	SS	11	
		3	SS	12	
111.0 / 2.3	**CLAYEY SILT**, sandy, trace gravel, very stiff, moist (GLACIAL TILL)	4	SS	24	
	...at 3.0 m, trace sand, hard	5	SS	39	
109.8 / 3.5	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)				...at 3.5m, auger grinding
		6	SS	50 / 50mm	
106.9 / 6.4					



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

file: 1-19-0719 bh logs.gpj

LOG OF BH3

Terraprobe

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

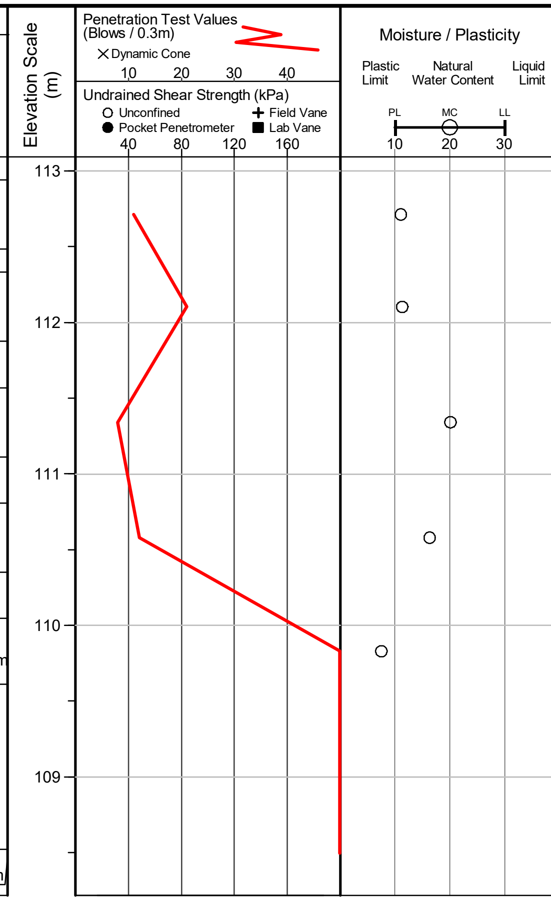
Position : E: 611305, N: 4825495 (UTM 17T)
Elevation Datum : Geodetic
Rig type : B-37
Drilling Method : Solid stem augers

SOIL PROFILE

Elev / Depth (m)	Description
113.1	GROUND SURFACE
	90mm **ASPHALTIC CONCRETE**
	50mm **AGGREGATE**
	FILL, clayey silt, trace sand, trace gravel, with shale fragments, firm to very stiff, greyish brown, moist
110.8 / 2.3	**CLAYEY SILT**, some sand, trace gravel, stiff, greyish brown, moist (GLACIAL TILL)
	...at 3.0 m, hard
109.6 / 3.5	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)
108.2 / 4.9	

SAMPLES

Number	Type	SPT 'N' Value	Lab Data and Comments: Grain Size Distribution (%) (MIT) GR SA SI CL
1	SS	11	
2	SS	21	
3	SS	8	
4	SS	12	4 13 53 30
5	SS	89 / 275mm	
6	SS	50 / 50mm	



Headspace Vapour (ppm) | Instrument Details | Unstabilized Water Level

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

LOG OF BH4

Terraprobe

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: 611306, N: 4825471 (UTM 17T) — Elevation Datum : Geodetic

Rig type : B-37 — Drilling Method : Solid stem augers

Depth Scale (m)	Elev / Depth (m)	Description (SOIL PROFILE)	Sample Number	Type	SPT 'N' Value	Elevation Scale (m)
0	112.5	**GROUND SURFACE**				
		90mm **ASPHALTIC CONCRETE**				
		FILL, clayey silt, trace sand, trace gravel, with shale fragments, organics, firm to stiff, wet	1	SS	6	112
1			2	SS	9	
	111.0 / 1.5	**CLAYEY SILT**, sandy, trace gravel, very stiff, greyish brown, moist (GLACIAL TILL)	3	SS	29	111
2	110.2 / 2.3	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)	4	SS	50 / 125mm	110
3	109.4 / 3.1		5	SS	50 / 50mm	

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

Penetration Test Values (Blows / 0.3m): ✕ Dynamic Cone — 10, 20, 30, 40

Undrained Shear Strength (kPa): O Unconfined, + Field Vane, ● Pocket Penetrometer, ■ Lab Vane — 40, 80, 120, 160

Moisture / Plasticity: Plastic Limit (PL), Natural Water Content (MC), Liquid Limit (LL) — 10, 20, 30

Headspace Vapour (ppm) | Instrument Details | Unstabilized Water Level | Lab Data and Comments — GRAIN SIZE DISTRIBUTION (%) (MIT): GR SA SI CL

file: 1-19-0719 bh logs.gpj

LOG OF BH5

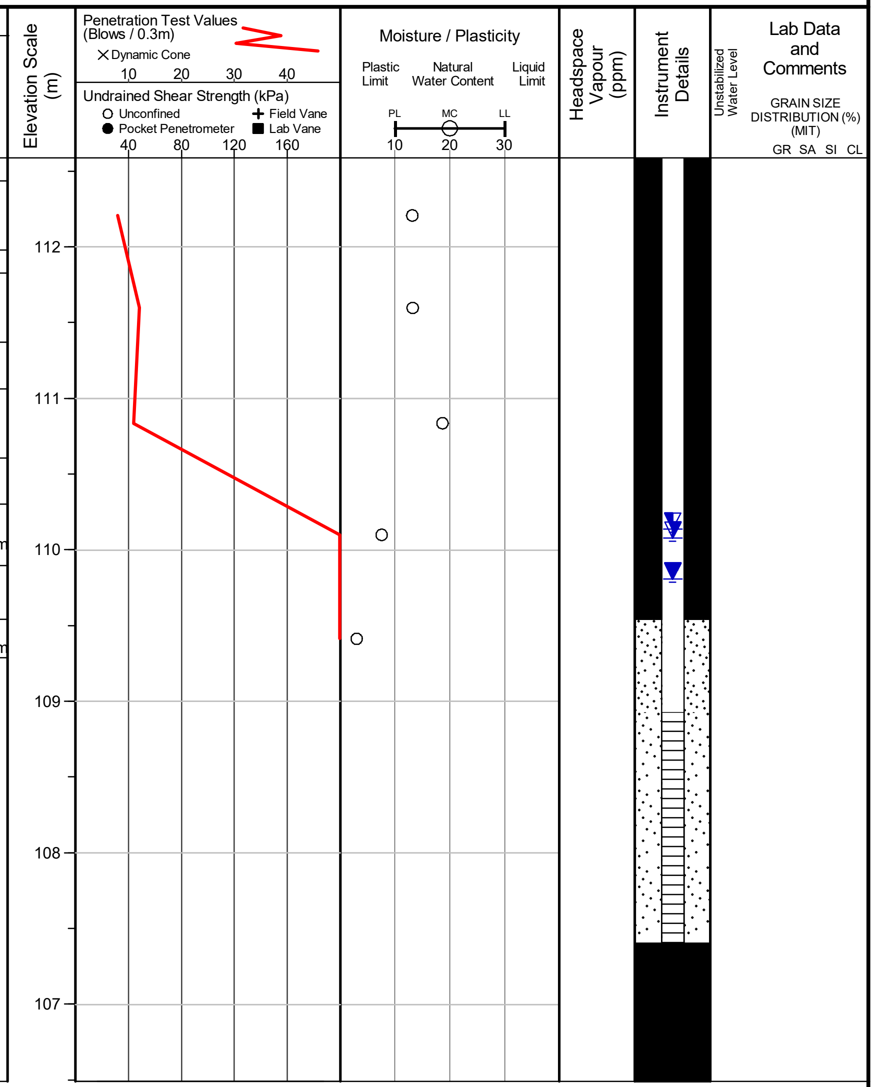
Terraprobe

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

Elev / Depth (m)	Description (SOIL PROFILE)	Sample Number	Type	SPT 'N' Value
112.6	GROUND SURFACE			
	90mm **ASPHALTIC CONCRETE**			
	FILL, clayey silt, trace sand, trace gravel, firm to stiff, greyish brown, moist	1	SS	8
		2	SS	12
111.4 / 1.2	**CLAYEY SILT**, sandy, trace gravel, trace shale fragments, stiff, grey, moist (GLACIAL TILL)	3	SS	11
	...at 2.3 m, some sand, hard	4	SS	89 / 250mm
109.6 / 3.0	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)	5	SS	50 / 100mm
106.5 / 6.1				



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

file: 1-19-0719 bh logs.gpj

LOG OF BH6

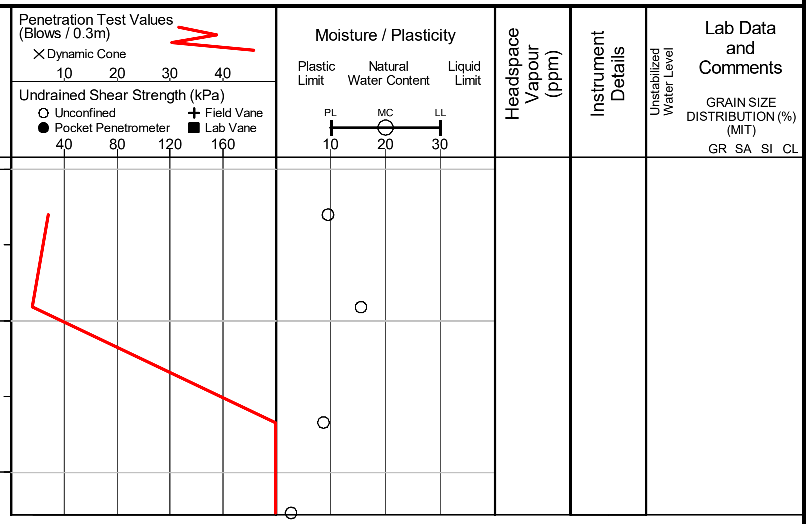
Terraprobe

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: 611327, N: 4825456 (UTM 17T) — Elevation Datum : Geodetic

Rig type : B-37 — Drilling Method : Solid stem augers

Depth Scale (m)	Elev / Depth (m)	Description (SOIL PROFILE)	Sample Number	Type	SPT 'N' Value	Elevation Scale (m)
0	112.1	**GROUND SURFACE**				112
		75mm **ASPHALTIC CONCRETE**				
		FILL, silty sand, trace clay, trace gravel, asphalt debris, loose, brown, moist	1	SS	7	
1		...at 0.8 m, clayey silt, trace sand, trace gravel, soft, grey, moist	2	SS	4	111
	110.6 / 1.5	**CLAYEY SILT**, sandy, trace gravel, hard, grey, moist (GLACIAL TILL)	3	SS	63	
2	109.8					110
	109.7 / 2.4	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)	4	SS	50 / 125mm	



END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

file: 1-19-0719 bh logs.gpj

Terraprobe

LOG OF BH7

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

Depth Scale (m)	Elev Depth (m)	SOIL PROFILE Description	Graphic Log	SAMPLES Number	Type	SPT 'N' Value	Elevation Scale (m)	Lab Data and Comments GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
0	111.6	**GROUND SURFACE**						
		75mm **ASPHALTIC CONCRETE**						
		FILL, sandy silt, some gravel, trace clay, trace asphalt debris, compact, dark grey, moist		1	SS	21	111	
1		...at 0.8 m, clayey silt, trace sand, trace gravel, stiff		2	SS	12		
	110.1 1.5	**CLAYEY SILT**, sandy, trace gravel, firm, grey, moist (GLACIAL TILL)		3	SS	7	110	
2		...at 2.3 m, trace sand, hard		4	SS	78 / 250mm	109	0 5 63 32
3	108.6 108.5 3.1	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)		5	SS	50 / 75mm		

Penetration Test Values (Blows / 0.3m)
× Dynamic Cone
10 20 30 40

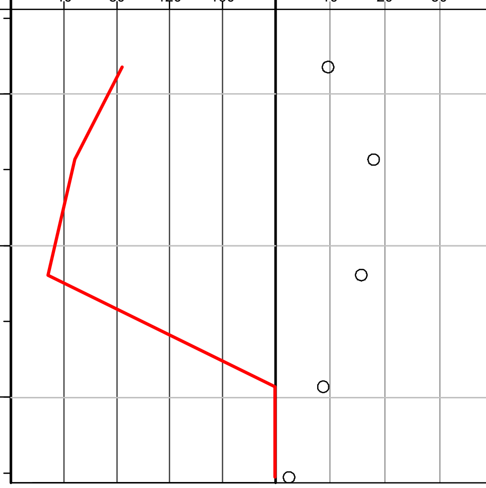
Undrained Shear Strength (kPa)
○ Unconfined ✚ Field Vane
● Pocket Penetrometer ■ Lab Vane
40 80 120 160

Moisture / Plasticity
Plastic Limit | Natural Water Content | Liquid Limit
PL MC LL
10 20 30

Headspace Vapour (ppm)

Instrument Details

Unstabilized Water Level



END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

file: 1-19-0719 bh logs.gpj

LOG OF BH8

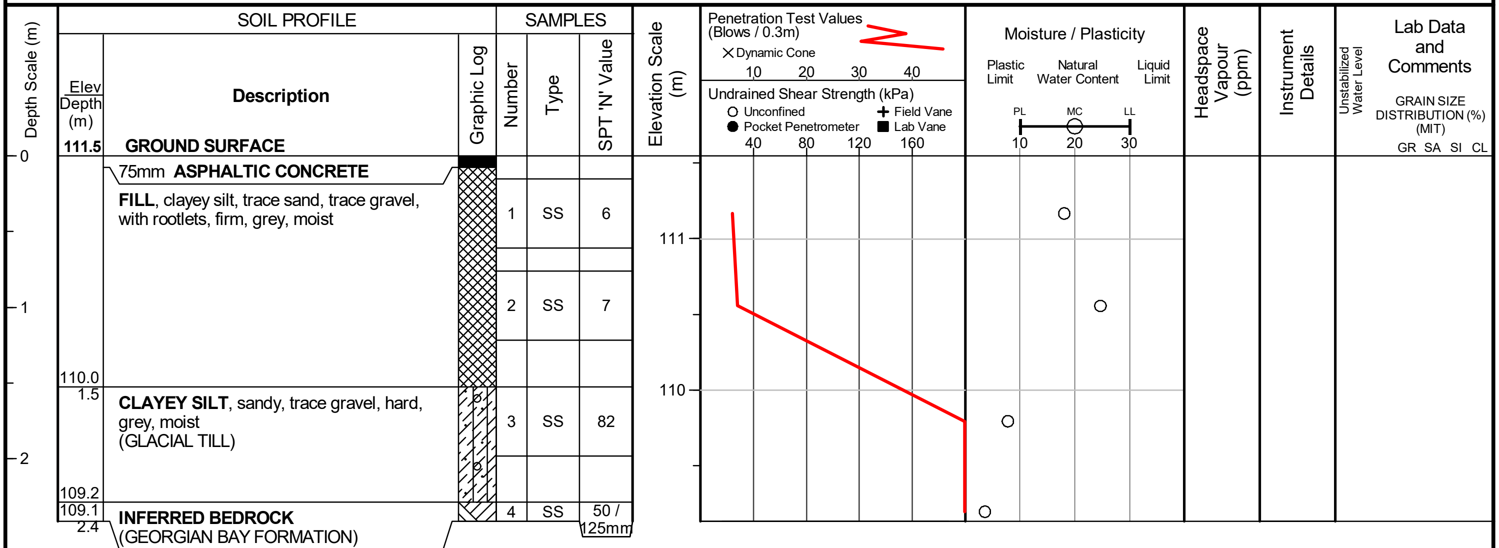
Terraprobe

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

Elev Depth (m)	Description	Number	Type	SPT 'N' Value	Natural Water Content
111.5	**GROUND SURFACE**				
	75mm **ASPHALTIC CONCRETE**				
	FILL, clayey silt, trace sand, trace gravel, with rootlets, firm, grey, moist	1	SS	6	
		2	SS	7	
110.0 1.5	**CLAYEY SILT**, sandy, trace gravel, hard, grey, moist (GLACIAL TILL)	3	SS	82	
109.2					
109.1 2.4	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)	4	SS	50 / 125mm	



END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

file: 1-19-0719 bh logs.gpj

LOG OF BH9

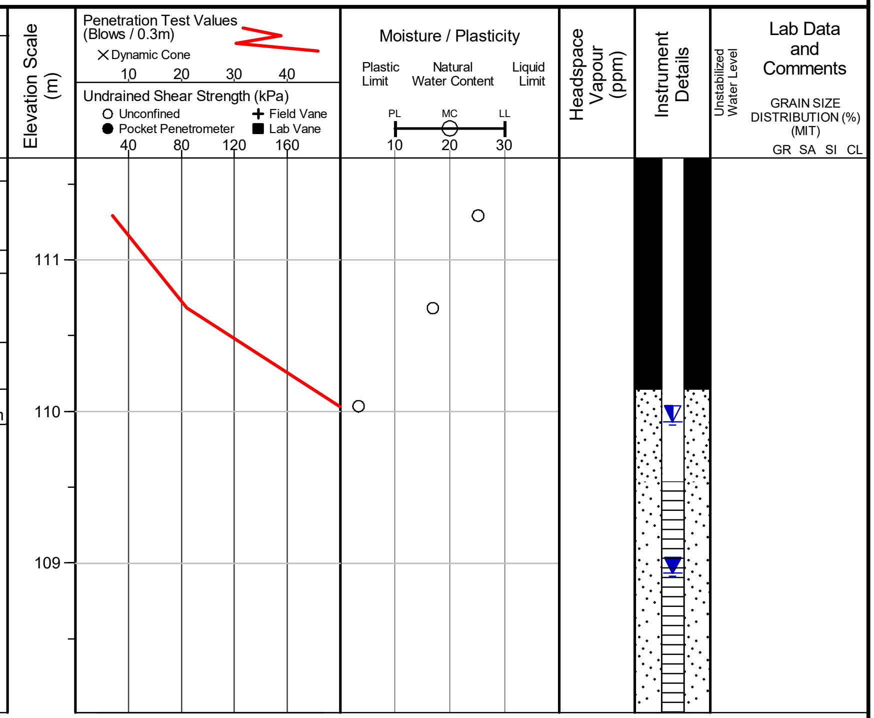
Terraprobe

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

Elev / Depth (m)	Description	Sample Number	Type	SPT 'N' Value
111.7	GROUND SURFACE			
	90mm **ASPHALTIC CONCRETE**			
	FILL, clayey silt, some sand, some gravel, with cinders, organics, firm, dark grey, moist	1	SS	7
110.9 / 0.8	**CLAYEY SILT**, sandy, trace gravel, very stiff, grey, moist (GLACIAL TILL)	2	SS	21
110.2 / 1.5	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)	3	SS	50 / 75mm
108.0 / 3.7	**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
Jan 9, 2020	2.7	108.9

Terraprobe

LOG OF BH10

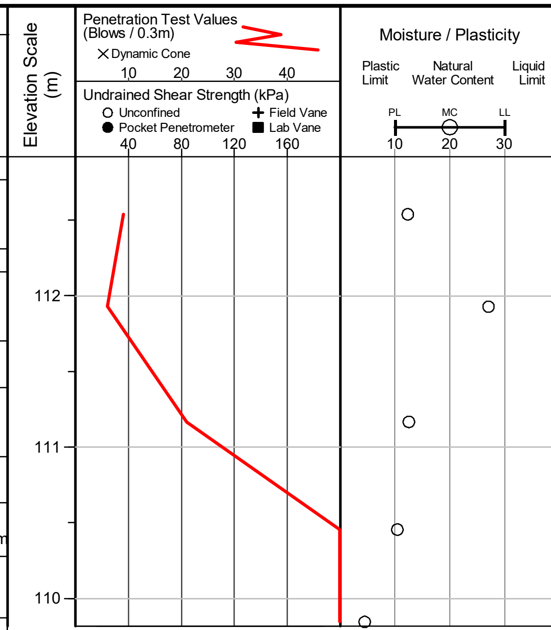
Project No.	: 1-19-0719-01	Client	: Ranee Management	Originated by	: DH
Date started	: December 3, 2019	Project	: 2570 - 2590 Argyle Road	Compiled by	: BV
Sheet No.	: 1 of 1	Location	: Mississauga, Ontario	Checked by	: SZ

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

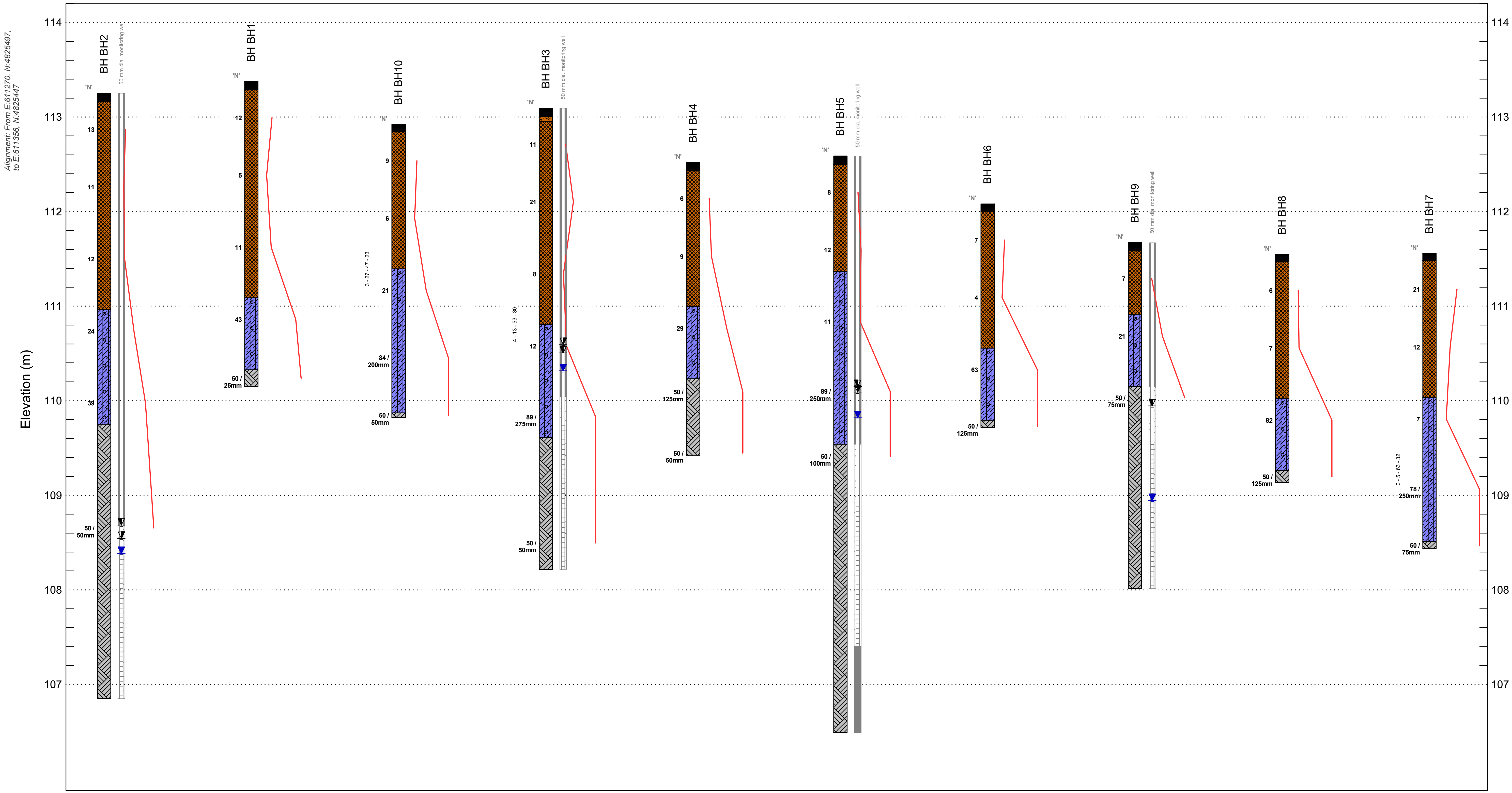
Elev / Depth (m)	Description (Soil Profile)	Sample Number	Type	SPT 'N' Value	Lab Data and Comments (Grain Size Distribution (%) (MIT): GR SA SI CL)
112.9	**GROUND SURFACE**				
	75mm **ASPHALTIC CONCRETE**				
	FILL, silty sand, trace gravel, trace clay, with brick debris, asphalt concrete, loose, brown, moist	1	SS	9	
	...at 0.8 m, clayey silt, firm, dark grey, wet	2	SS	6	
111.4 / 1.5	**CLAYEY SILT**, sandy, trace gravel, very stiff, greyish brown, moist (GLACIAL TILL)	3	SS	21	3 27 47 23
	...at 2.3 m, trace sand, hard, grey, damp	4	SS	84 / 200mm	
109.9 / 109.8 / 3.1	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)	5	SS	50 / 50mm	

END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

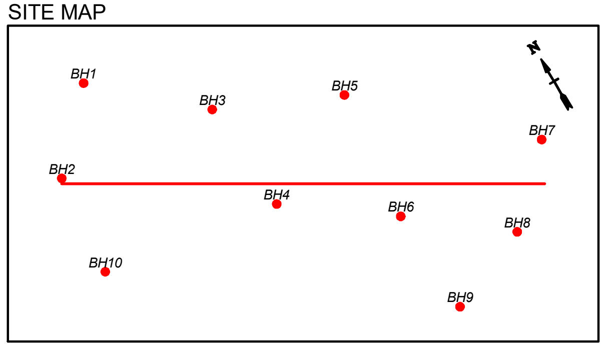


file: 1-19-0719 bh logs.gpj



Boreholes Equally Spaced

SITE MAP



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

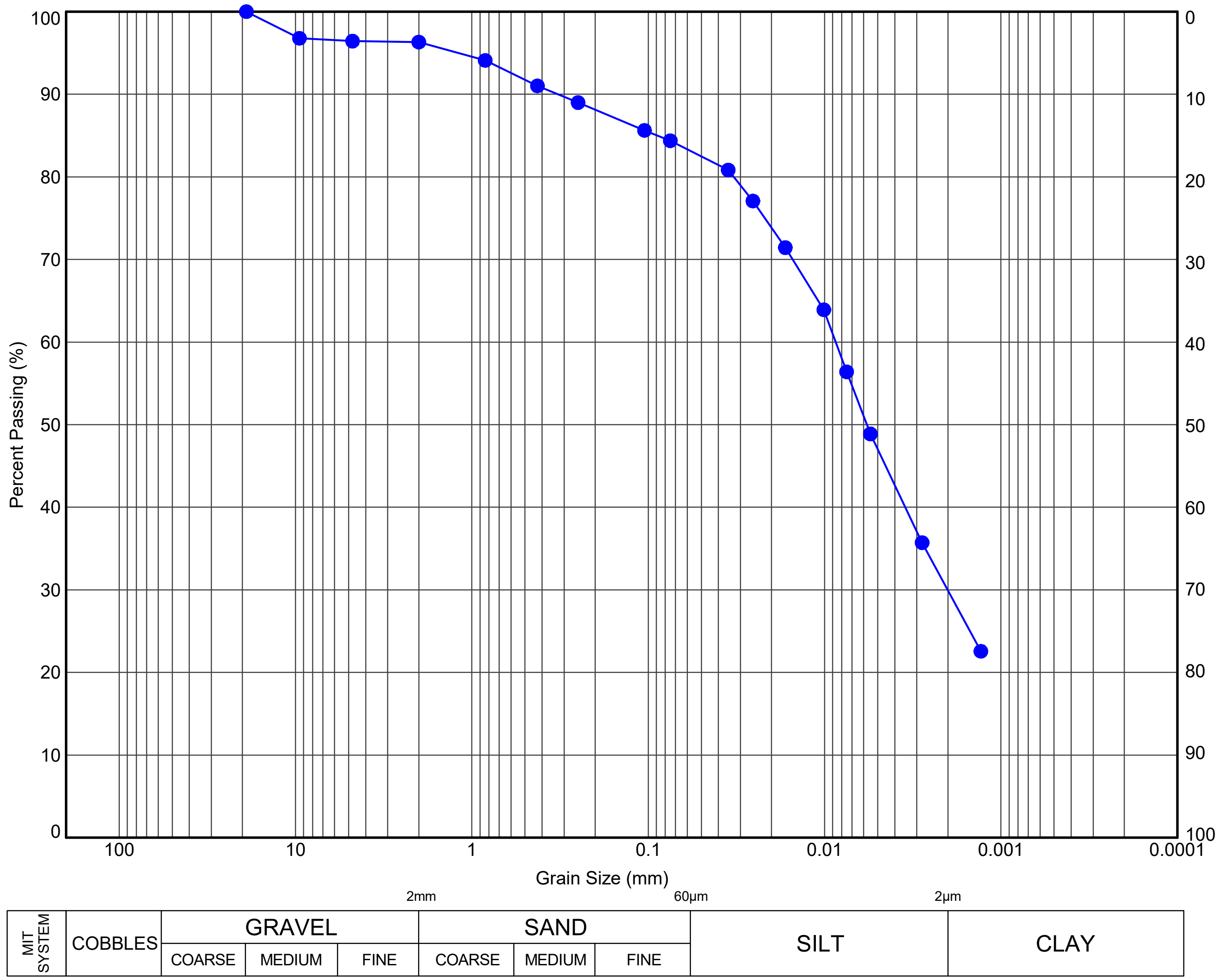
Terraprobe
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title: **SUBSURFACE PROFILE**

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

APPENDIX B

TERRAPROBE INC.



MIT SYSTEM

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

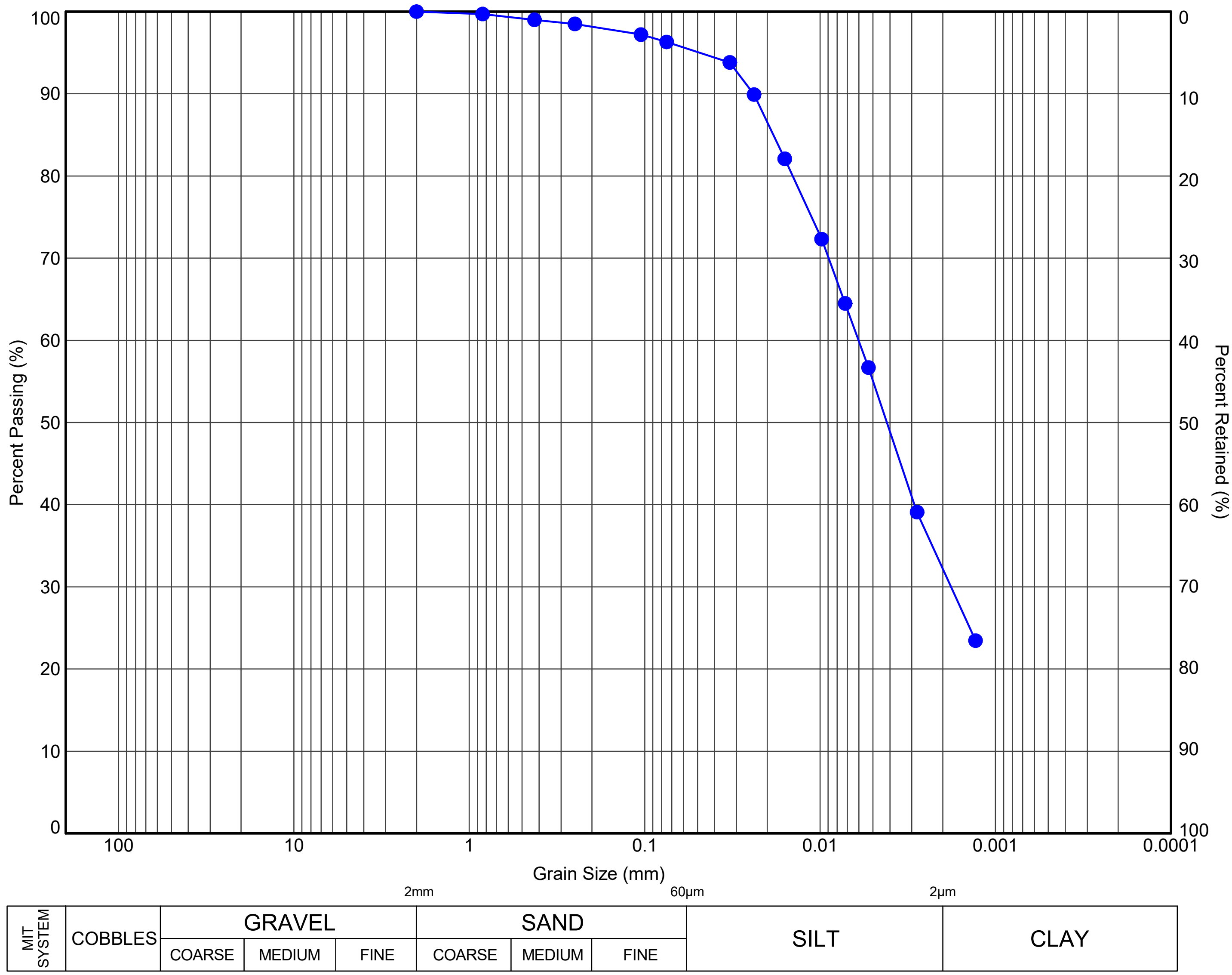
Terraprobe

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

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

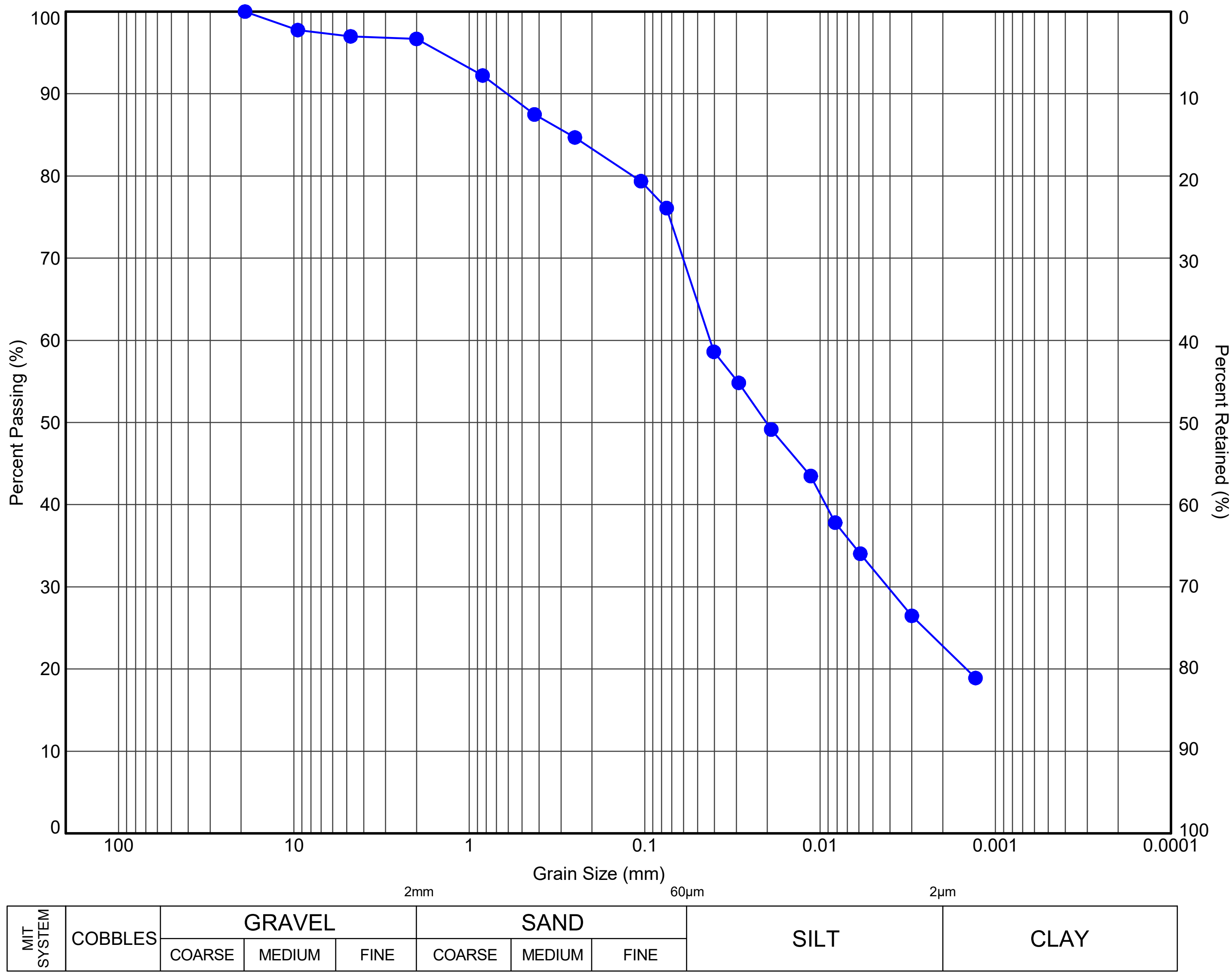
Terraprobe

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

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

Terraprobe
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

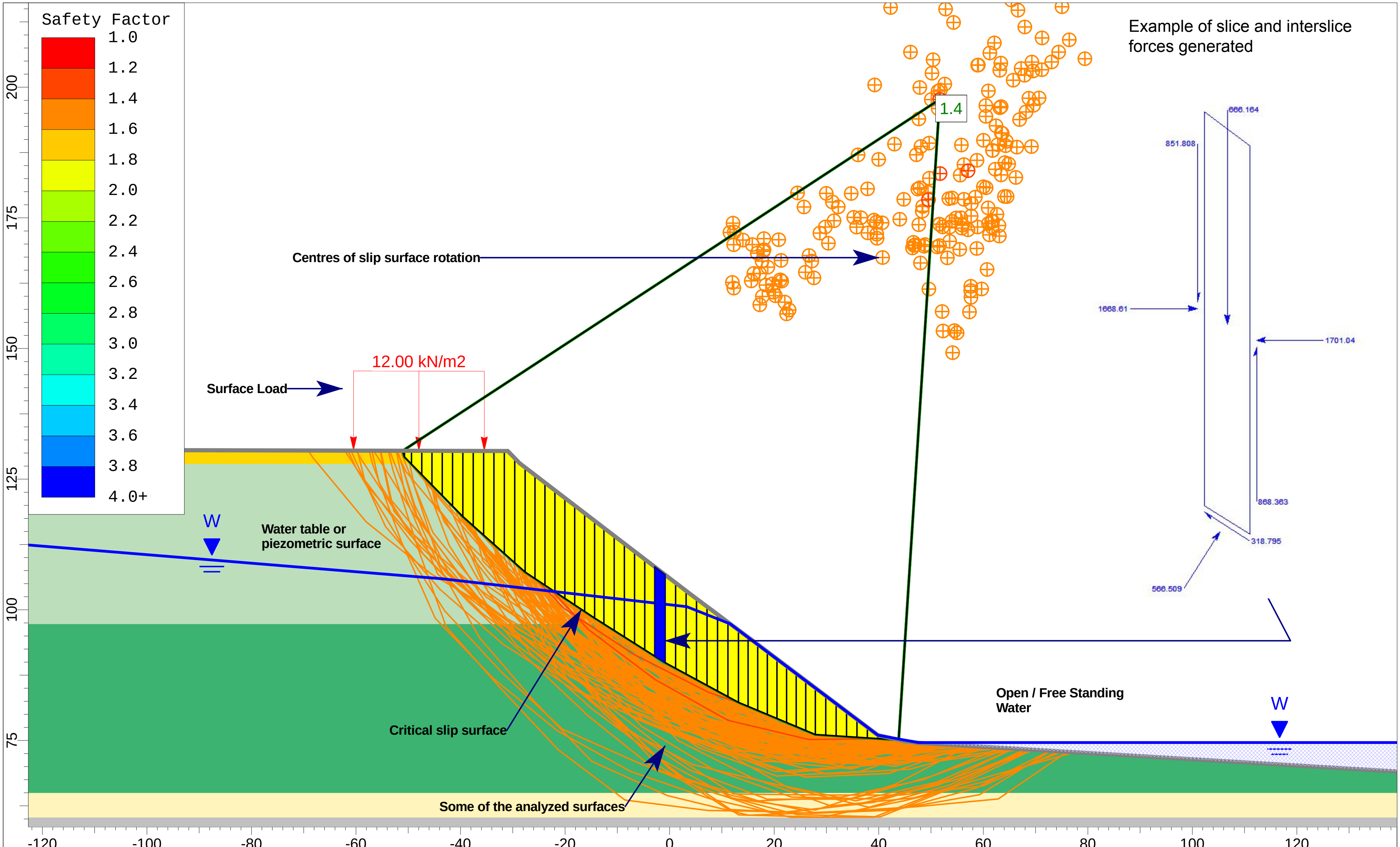
Location: Slope toe near Section D-D'

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.



Terraprobe

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Construction Materials Inspection & Testing

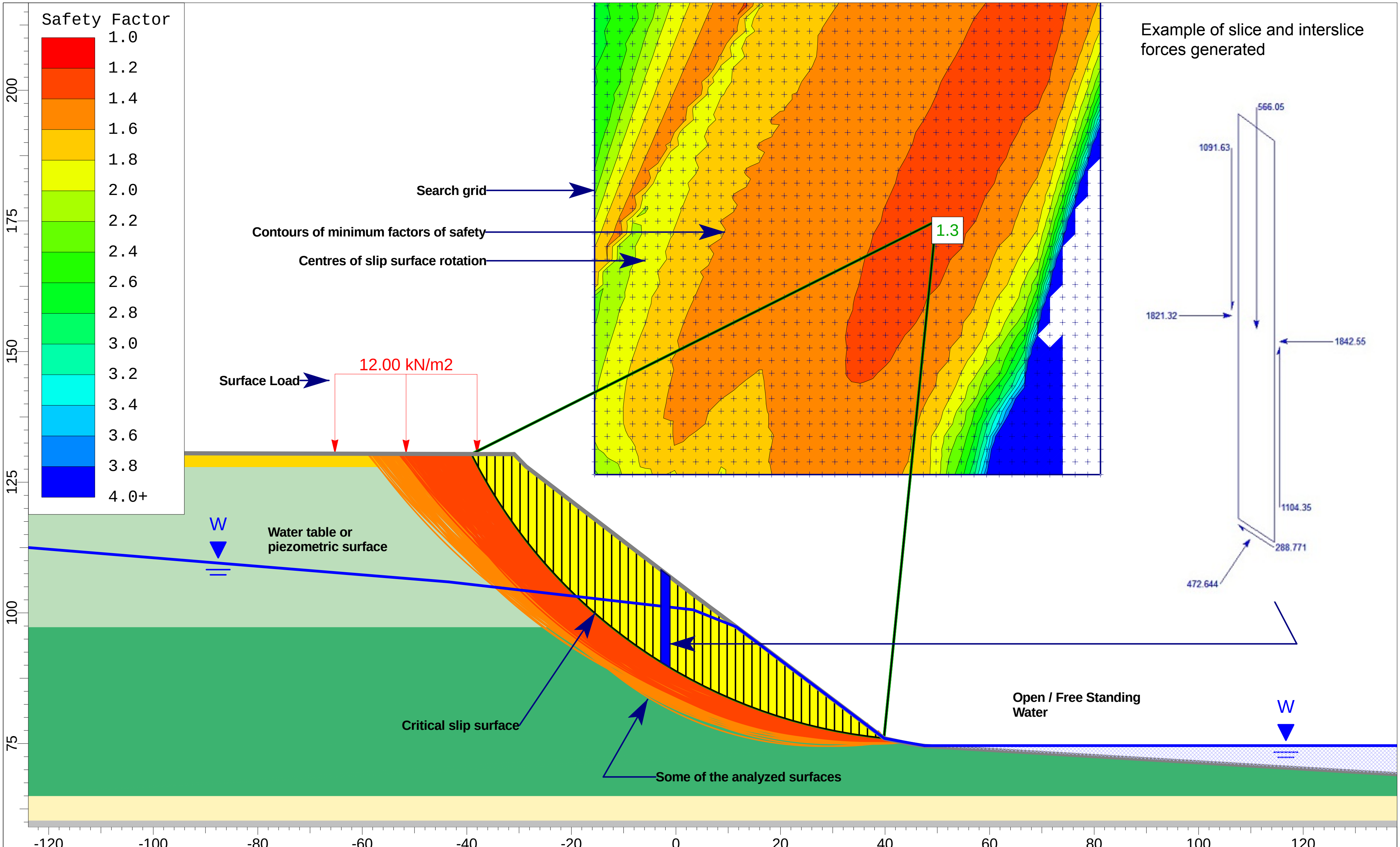
SLIDEINTERPRET 6.026

Project

Slope Stability Analysis - Explanation with Slices and Slice Forces

Analysis Description

Non-Circular Analysis (example)



Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

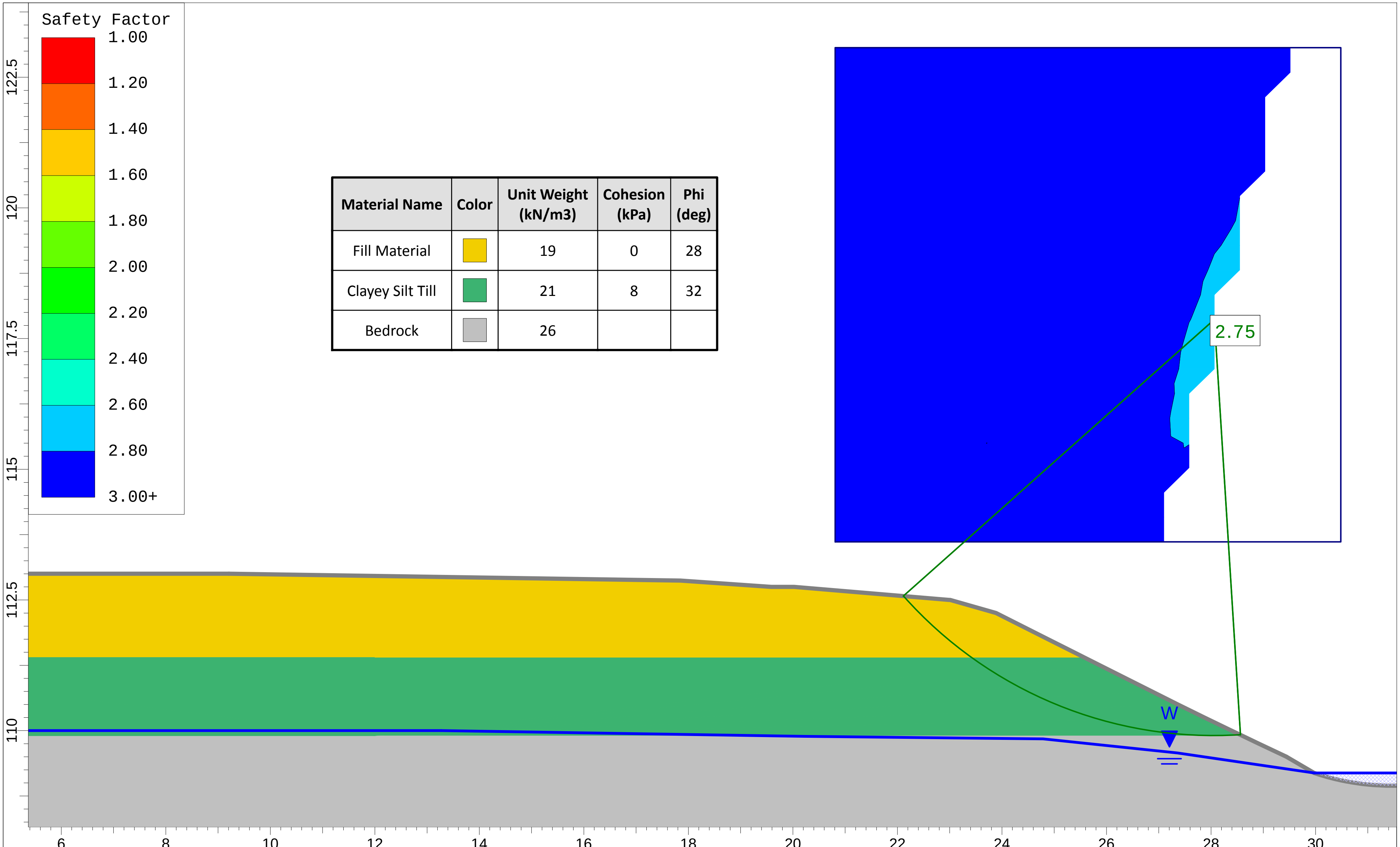
SLIDEINTERPRET 6.026

Project

Slope Stability Analysis - Explanation with Slices and Slice Forces

Analysis Description

Circular Analysis (example)



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

Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

SLIDEINTERPRET 7.031

Notes

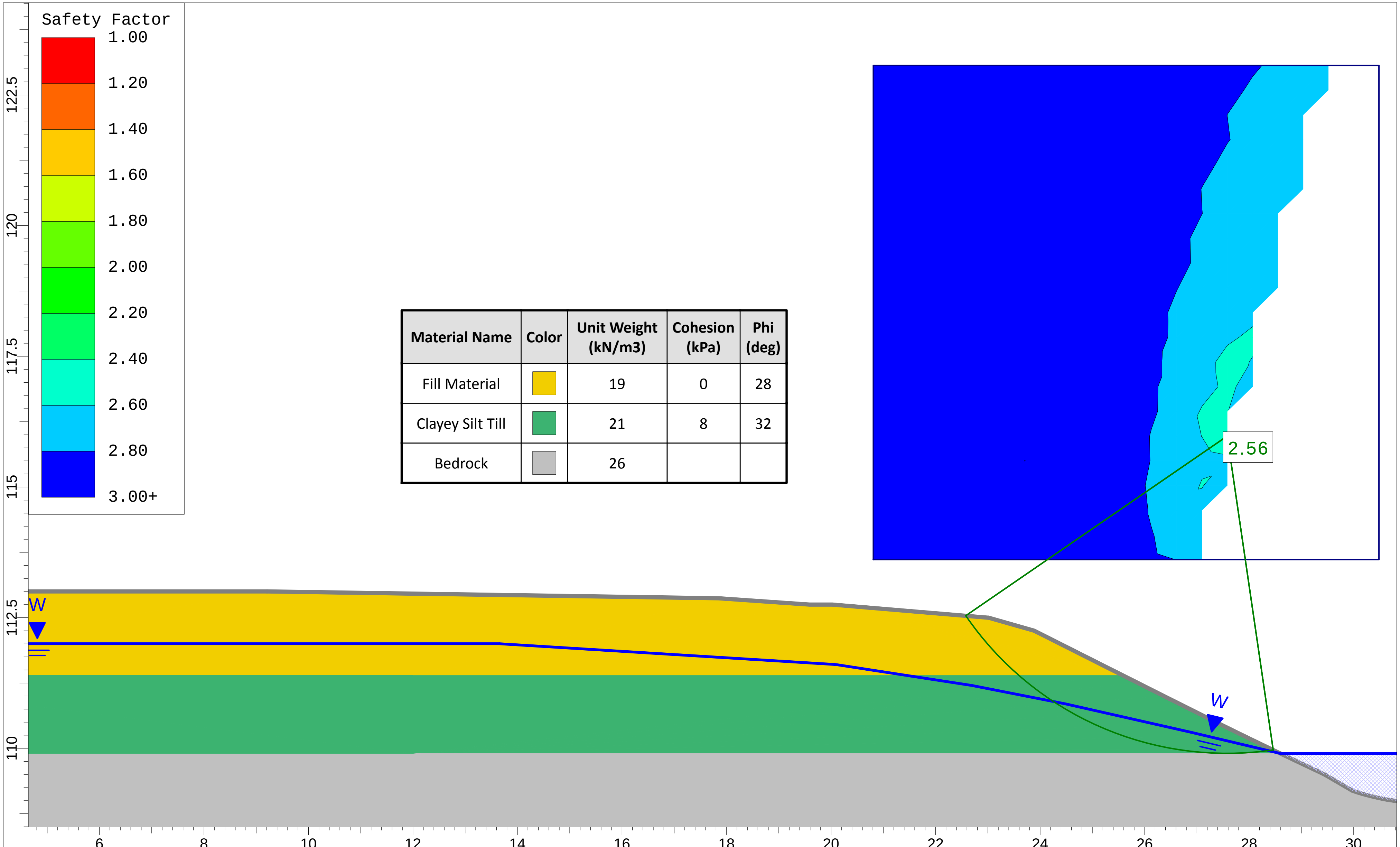
Refer to appended Slope Stability Analysis Explanation sheets for legend. Refer to cross-sections for inclinations and other pertinent slope information.

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

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

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

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



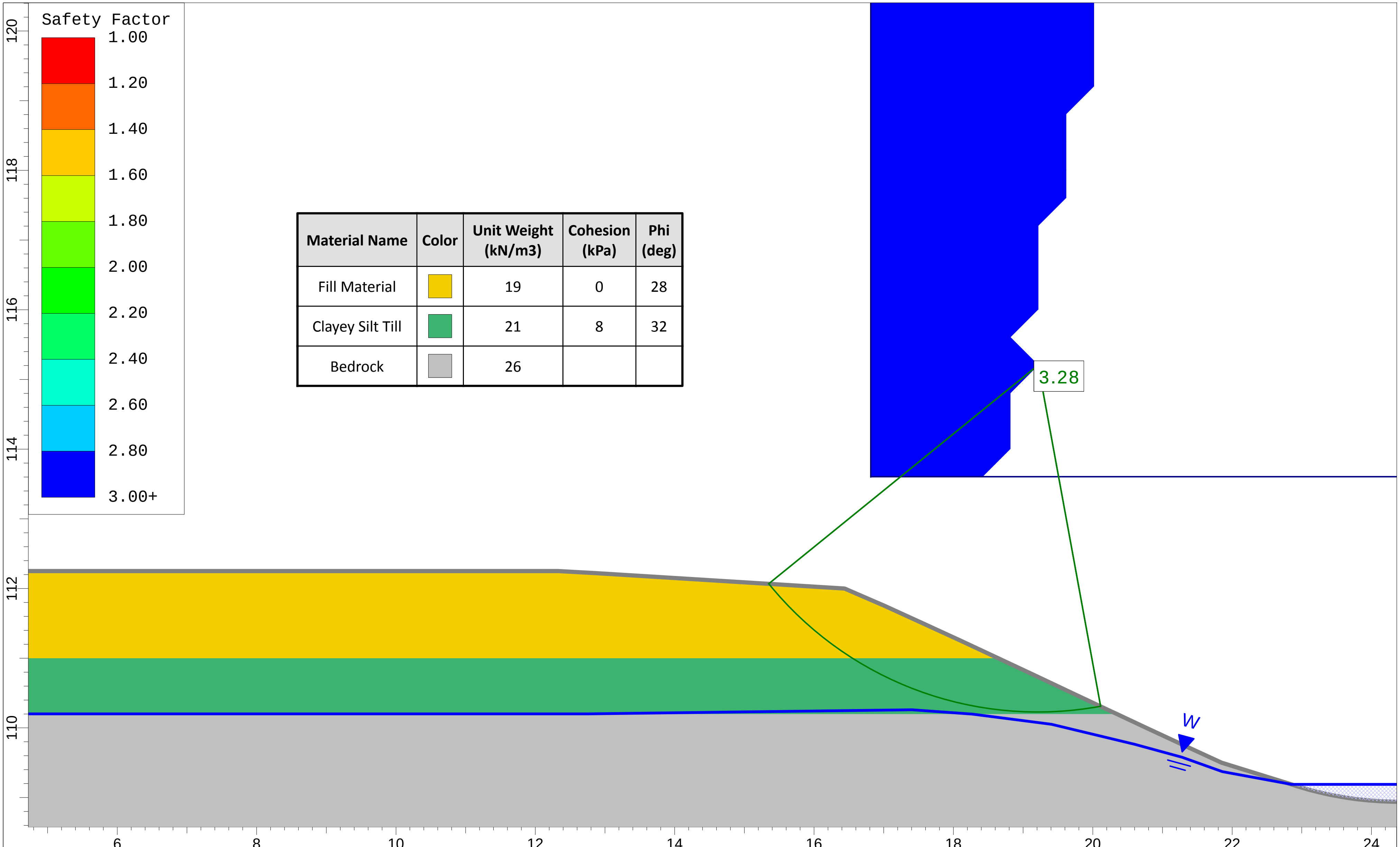
Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Fill Material		19	0	28
Clayey Silt Till		21	8	32
Bedrock		26		

Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

SLIDEINTERPRET 7.031

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, HWL
Date	5/21/2020
Scale	1:100
File	2570-2590 Argyle Road.slmd
By	BV
Ref.	Topographic Survey from Speight, Van Nostrand & Gibson Ltd.



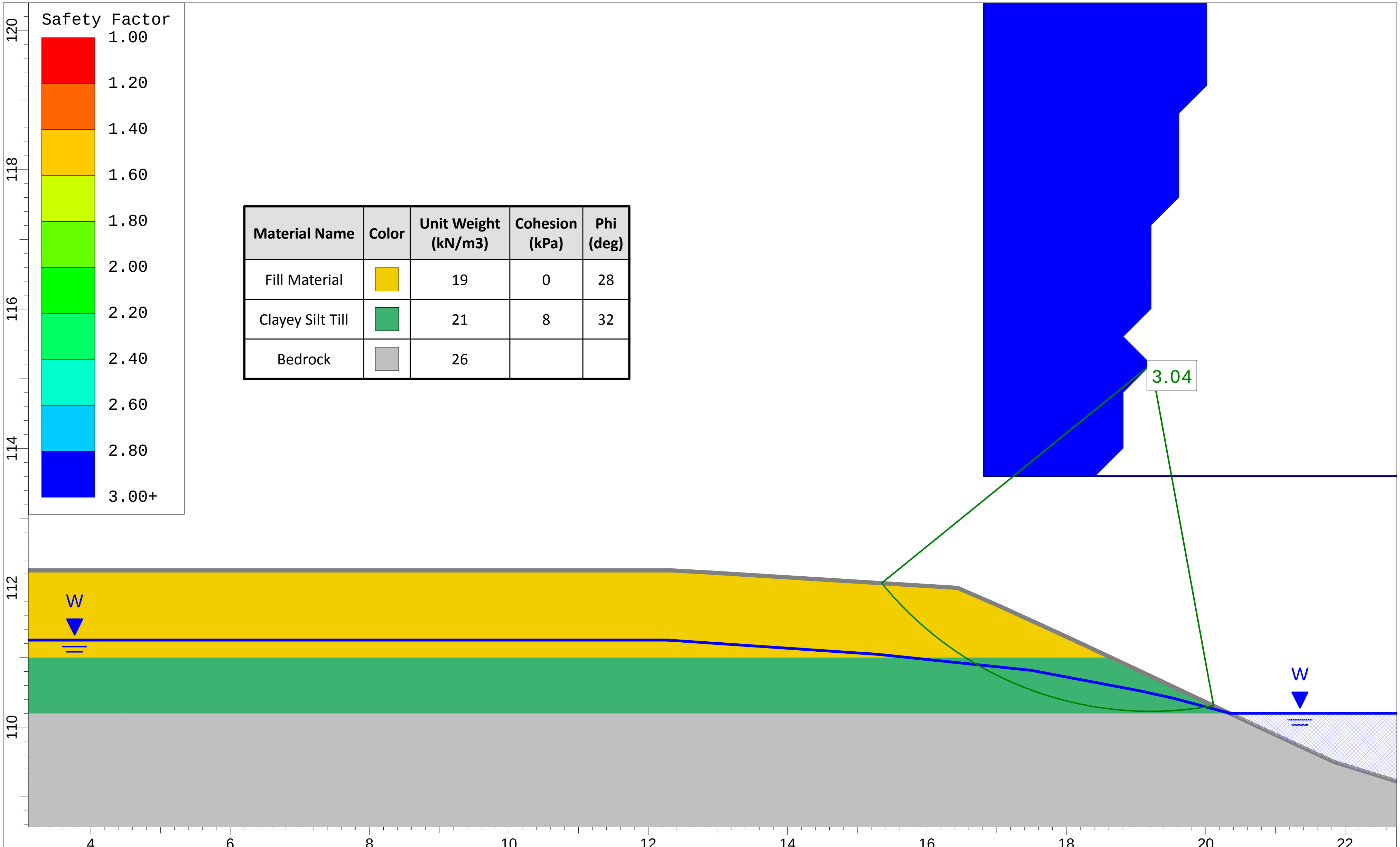
Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Fill Material		19	0	28
Clayey Silt Till		21	8	32
Bedrock		26		

Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

SLIDEINTERPRET 7.031

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 B-B', Existing Conditions, NWL
Date	5/21/2020
Scale	1:75
File	2570-2590 Argyle Road.slmd
By	BV
Ref.	Topographic Survey from Speight, Van Nostrand & Gibson Ltd.



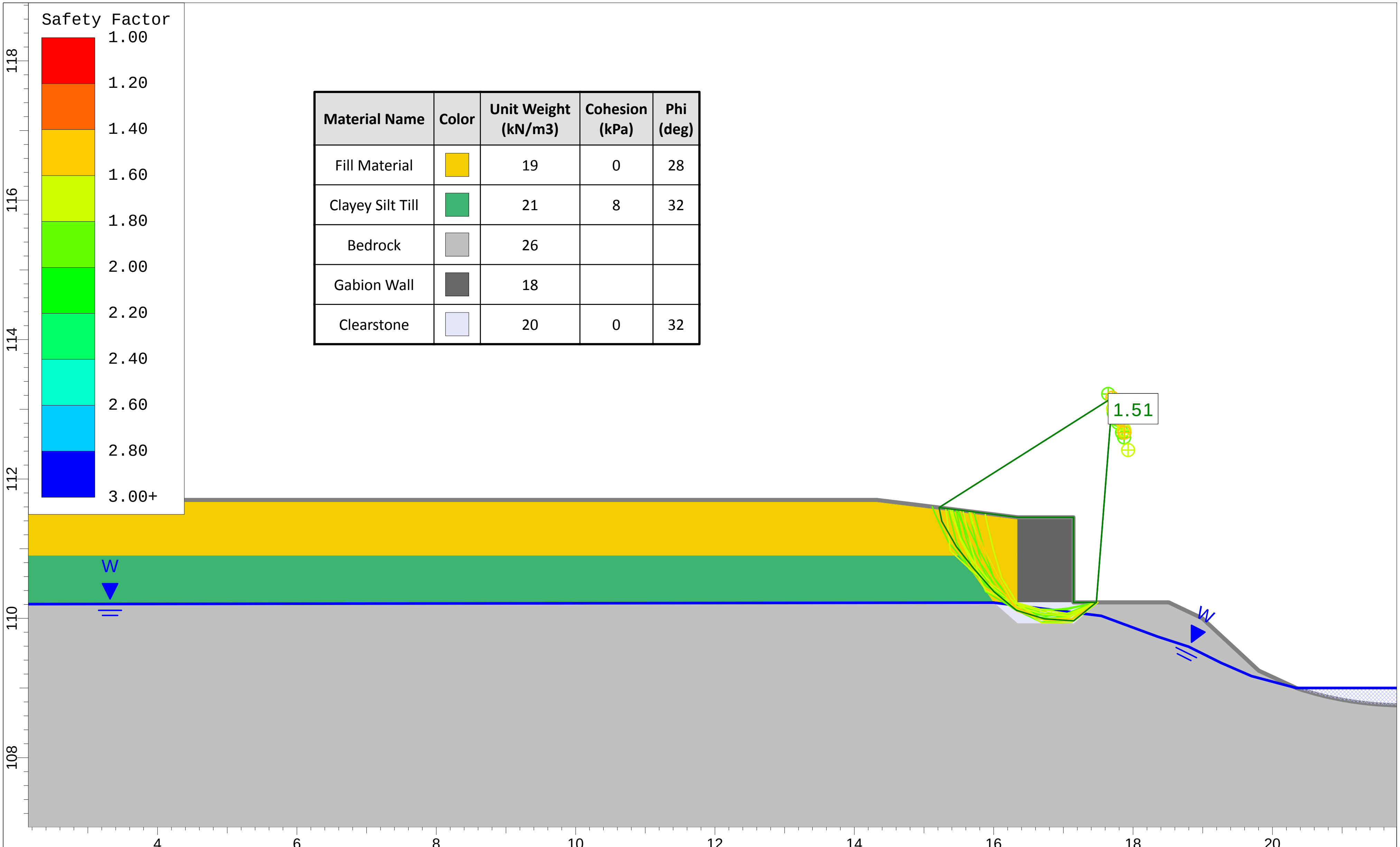
Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Fill Material		19	0	28
Clayey Silt Till		21	8	32
Bedrock		26		

Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

SLIDEINTERPRET 7.031

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 B-B', Existing Conditions, HWL
Date	5/21/2020
Scale	1:75
File	2570-2590 Argyle Road.slmd
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

Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

SLIDEINTERPRET 7.031

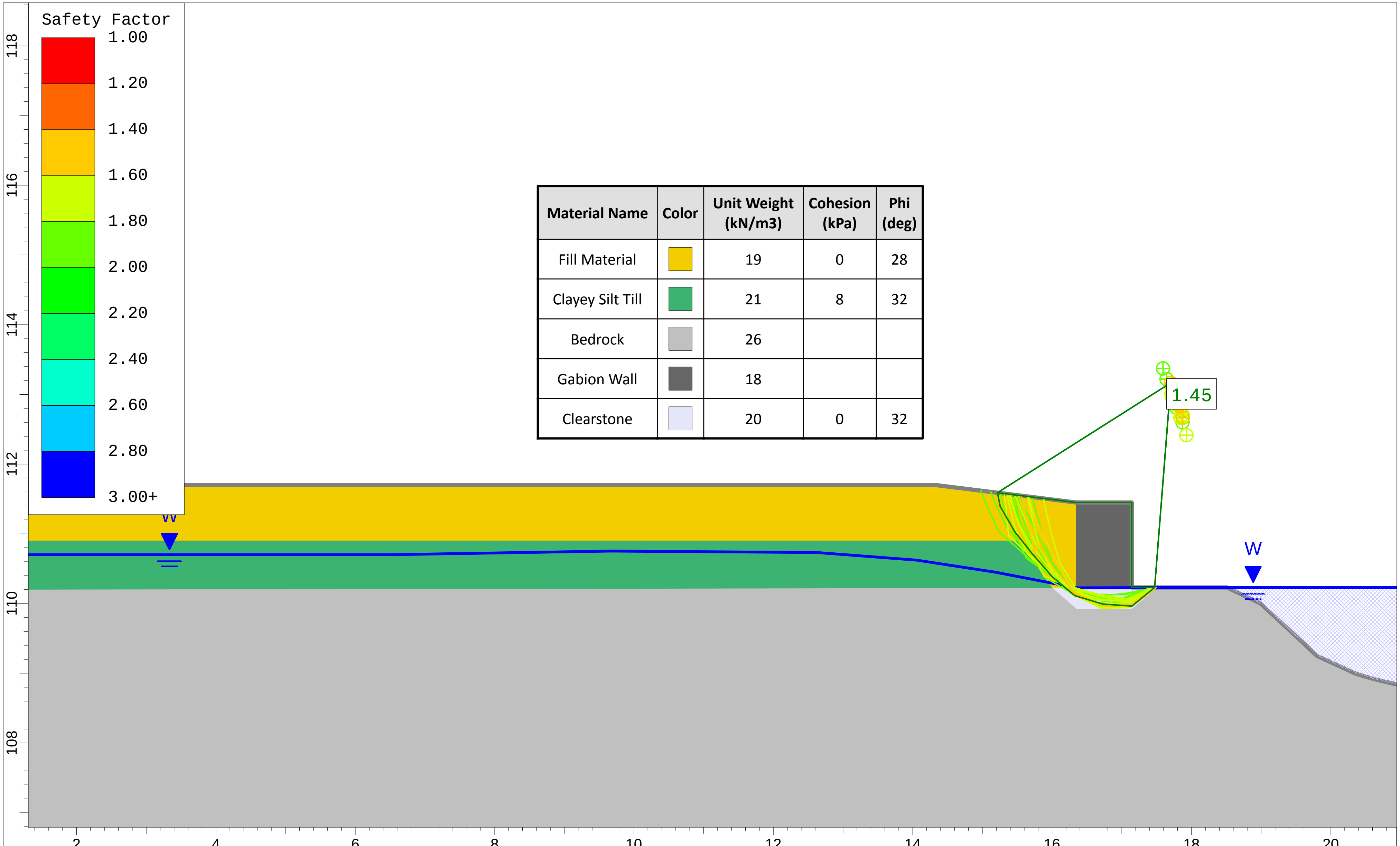
Notes
Refer to appended Slope Stability Analysis Explanation sheets for legend. Refer to cross-sections for inclinations and other pertinent slope information.

Project: **1-19-0719-01 2570-2590 Argyle Road**

Analysis: Slope Stability: Section C-C', Existing Conditions, NWL

Date	Scale	File
5/21/2020	1:75	2570-2590 Argyle Road.slmd

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



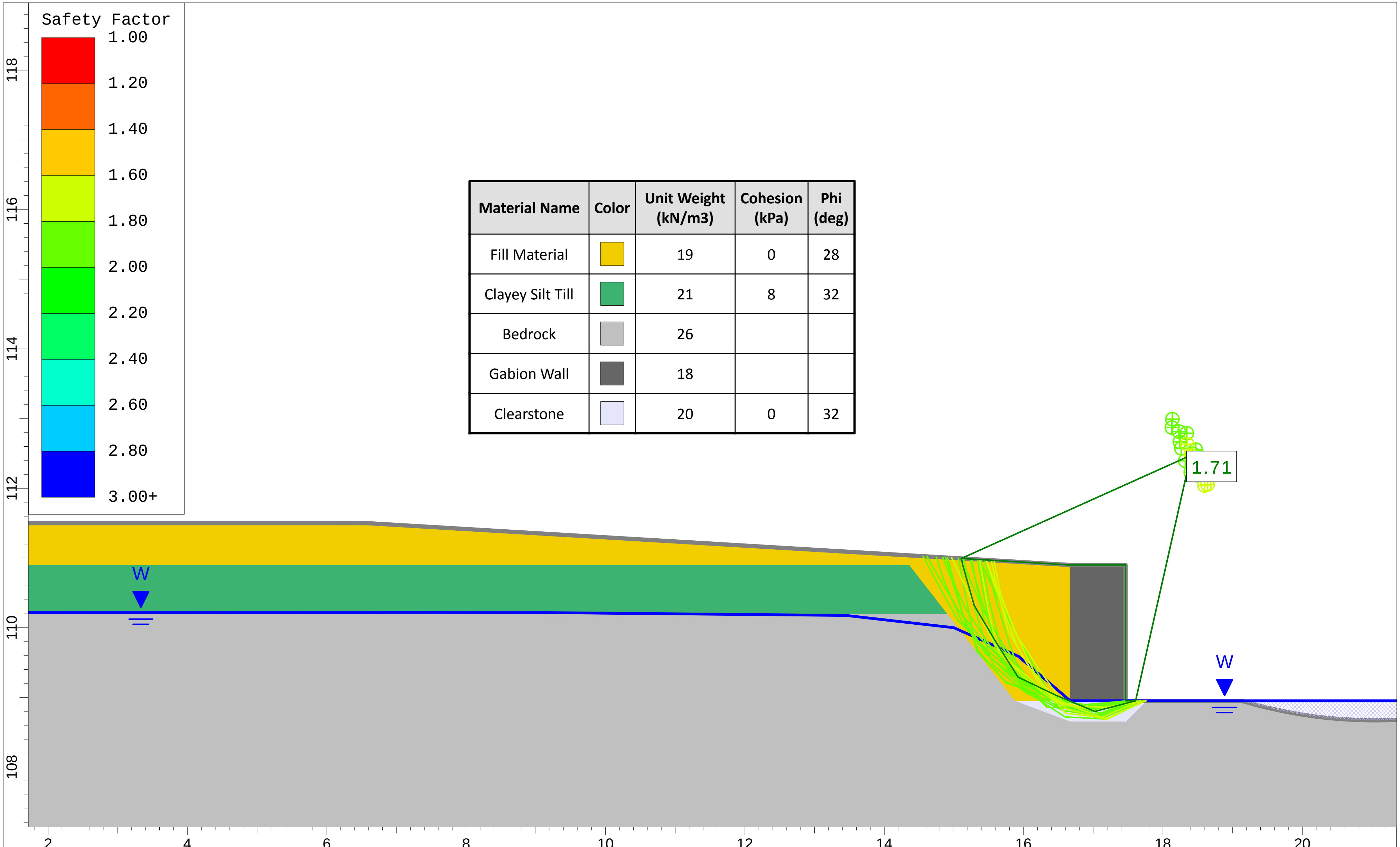
Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Fill Material		19	0	28
Clayey Silt Till		21	8	32
Bedrock		26		
Gabion Wall		18		
Clearstone		20	0	32

Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

SLIDEINTERPRET 7.031

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, HWL
Date	5/21/2020
Scale	1:75
File	2570-2590 Argyle Road.slmd
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

Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

SLIDEINTERPRET 7.031

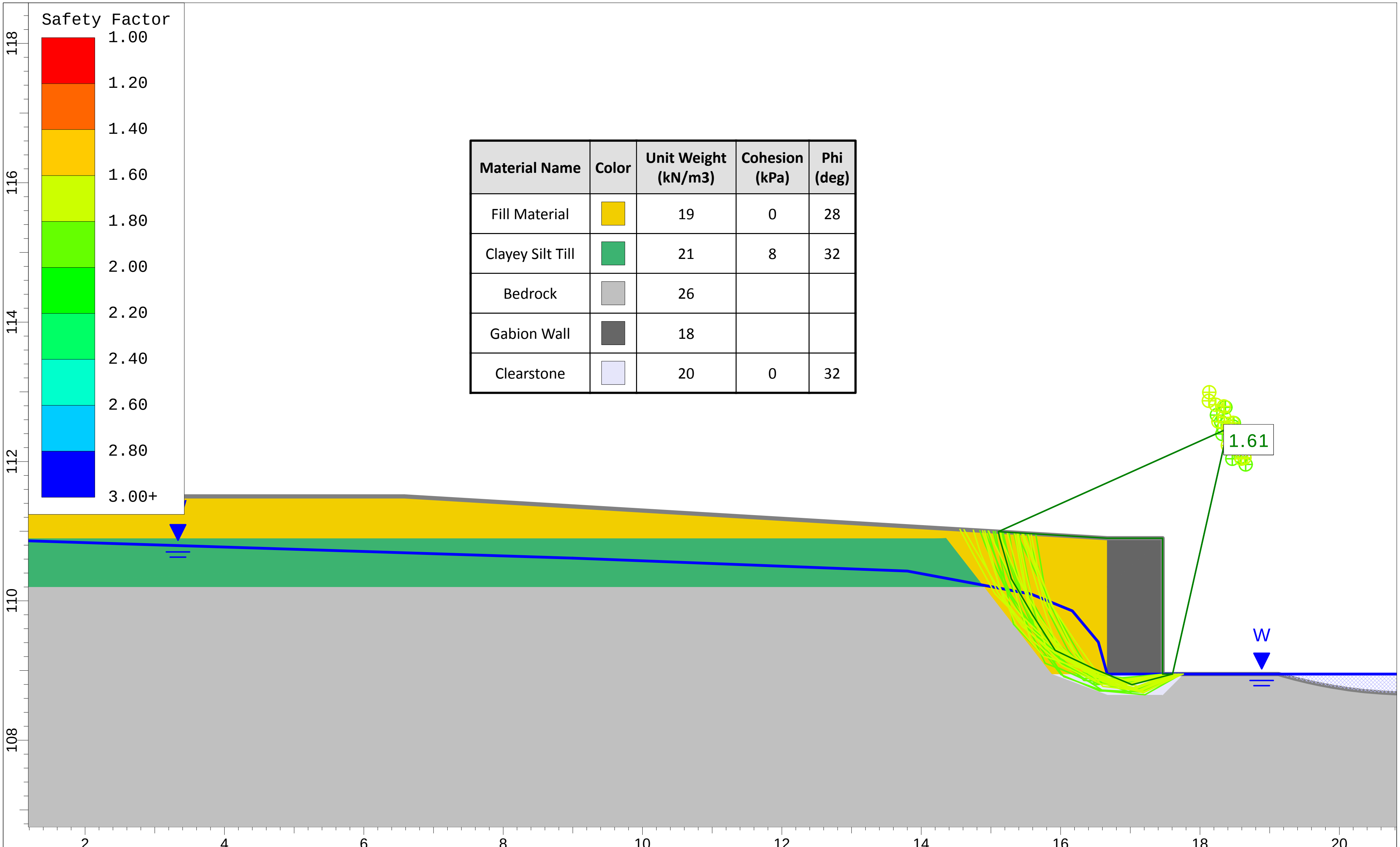
Notes
Refer to appended Slope Stability Analysis Explanation sheets for legend. Refer to cross-sections for inclinations and other pertinent slope information.

Project: **1-19-0719-01 2570-2590 Argyle Road**

Analysis: Slope Stability: Section D-D', Existing Conditions, NWL

Date	Scale	File
5/21/2020	1:75	2570-2590 Argyle Road.slmd

By	Ref.
BV	Topographic Survey from Speight



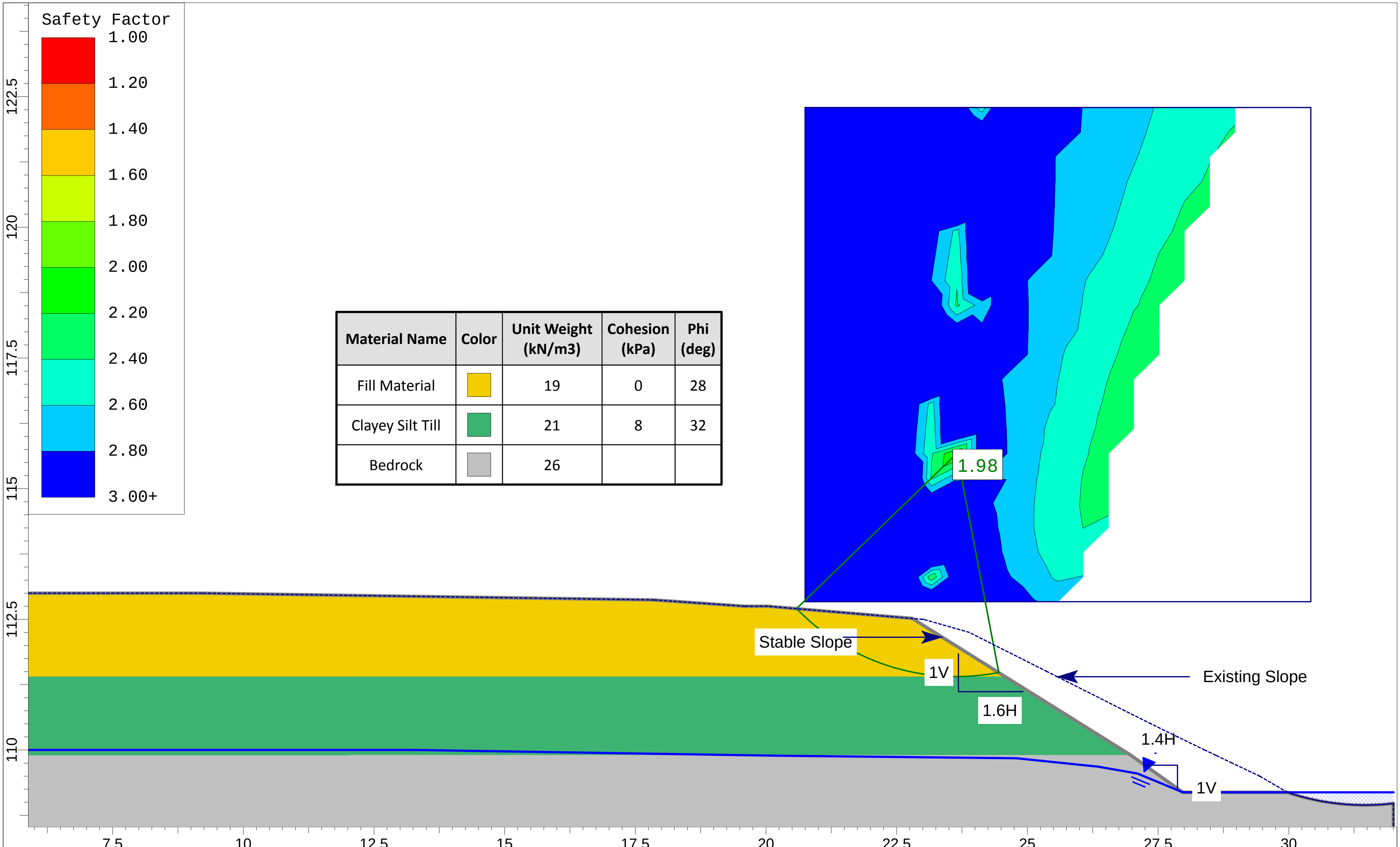
Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Fill Material		19	0	28
Clayey Silt Till		21	8	32
Bedrock		26		
Gabion Wall		18		
Clearstone		20	0	32

Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

SLIDEINTERPRET 7.031

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.slmd
By	BV
Ref.	Topographic Survey from Speight



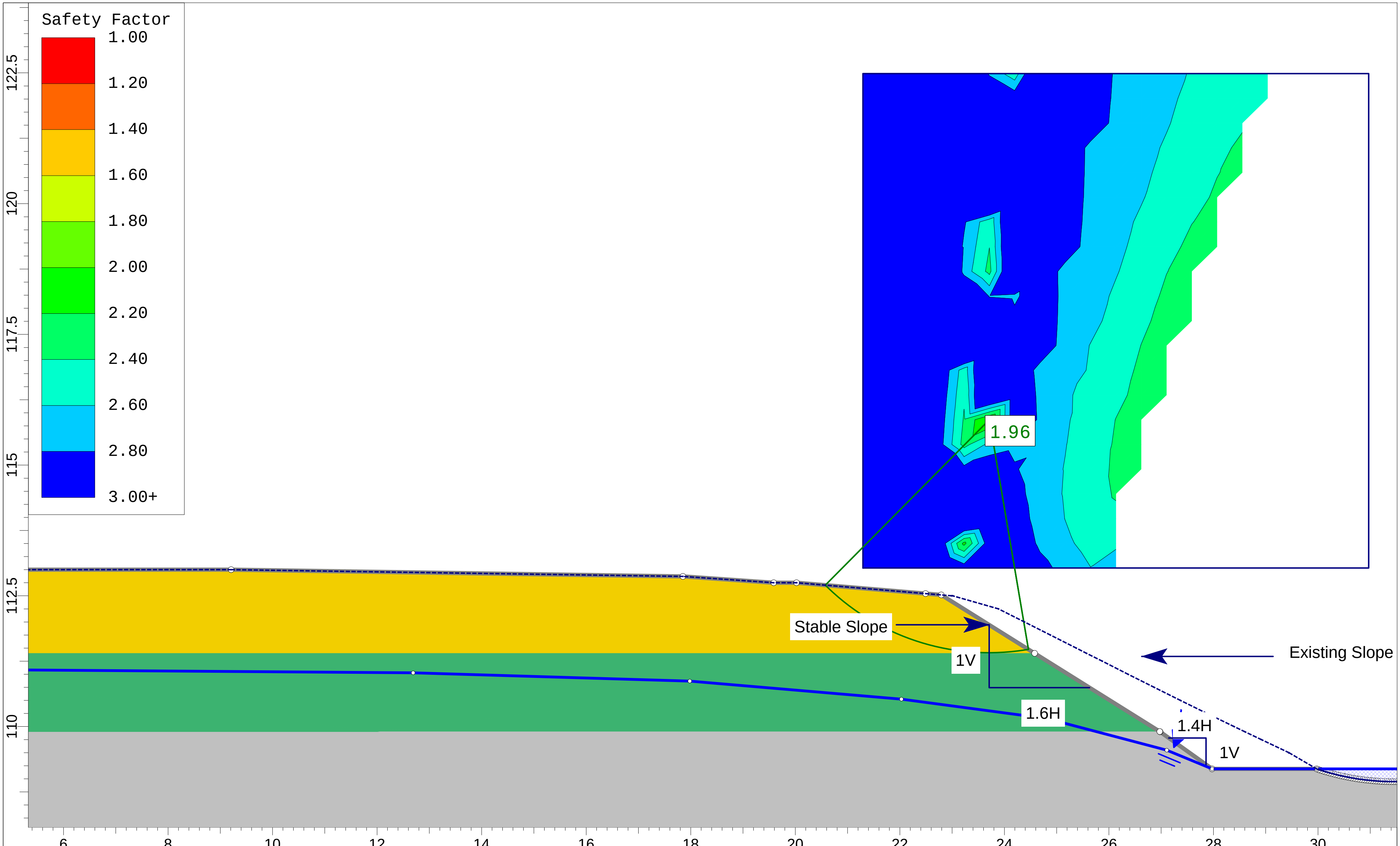
Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Fill Material		19	0	28
Clayey Silt Till		21	8	32
Bedrock		26		

Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

SLIDEINTERPRET 7.031

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, NWL
Date	5/21/2020
Scale	1:100
File	2570-2590 Argyle Road.slmd
By	BV
Ref.	Topographic Survey from Speight, Van Nostrand & Gibson Ltd.



Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

SLIDEINTERPRET 7.031

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