Design of geosynthetic reinforced slopes in cohesive soils: theory and example case studies

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ABSTRACT
The current design practice of geosynthetic reinforcements assume the backfill to be purely frictional irrespective of soil type. However, recent editions of AASHTO LRFD bridge design specifications allow for the inclusion of cohesion in the design of geo-reinforced slopes. Also recent studies have shown that accounting for the presence of even a modest amount of cohesion may allow using locally available cohesive backfills to a greater extent and less overall reinforcement. Nevertheless, practitioners need to be aware that unlike purely frictional backfills, cohesive soils are subject to the formation of tension cracks. These cracks make the slope less stable and therefore need to be properly accounted for in any geo-reinforcement design relying on soil cohesion.
Abd and Utili (2017) derived a semi-analytical method for uniform c - phi slopes accounting for the formation of tension cracks that provides the amount of reinforcement needed as a function of soil cohesion, tensile strength, angle of shearing resistance and slope inclination by means of the kinematic approach of limit analysis.
In this paper first a summary of the main tenets of the theoretical formulation to design the reinforcement is laid out, then several example case studies are presented to illustrate the application of the new theory to various cohesive backfills representative of locally available soils in Brazil. Calculations of the required reinforcement strength are carried out for both uniform and linearly increasing reinforcement distributions. Also, the economic savings that can be achieved by accounting for the presence of cohesion are estimated.

KEYWORDS: Geosynthetics; Upper bound limit analysis; Kinematic approach; Cohesive soils; Tension cracks

1. INTRODUCTION

In tropical regions, such as Brazil, there is a large number of fine-grained soils that have been subjected to the process of laterization. Unlike cohesive soils from temperate regions, these materials exhibit low plasticity and compressibility, and high strength characteristics. Care should be taken to guarantee proper drainage. This can be achieved by the use of permeable reinforcements, such as nonwoven geotextiles (Zornberg and Mitchell, 1994). Therefore, the use of cohesive soils as backfill can lead to significant savings in transportation costs in several tropical countries and in general in regions where high quality granular fills are not easily available. Unfortunately, most of the design methods for reinforced slopes currently available in the literature, primarily limit equilibrium based methods, deal only with cohesionless soils (de Buhan et al., 1989; Leshchinsky and Boedeker, 1989; Jewell, 1991; Leshchinsky et al., 1995; Michalowski, 1997). On the other hand, working stress design methods have been developed in the past years in an attempt to better predict reinforced soil structures behaviour at the service limit state, with some of them accounting for the beneficial effect of cohesion in their formulations (Ehrlich and Mitchell, 1994; Dantas and Ehrlich, 2000; Miyata and Bathrust, 2007; Bathrust et al., 2008; Allen and Bathrust, 2015). Therefore research is needed to extend the inclusion of cohesion for the design at the ultimate limit state of slope reinforcement which is the aim of this paper.

Several authors consider well-graded granular soils as more appropriate for the use in GRS-RW due to their good mechanical properties in terms of angle of shearing resistance and drainage (Schlosser and Delage, 1987; Jones, 1996). However, Guler et al. (2007) and Zornberg and Mitchell (1994) highlight that in regions where clean granular materials are difficult to retrieve or too expensive, the use of cohesive soils can be advantageous. In fact, there is a number of cases of geosynthetic reinforced soil structures, especially with non-woven geotextiles and geogrids, constructed with cohesive soils that have shown successful performance in field trials (Tatsuoka et al., 1998; Riccio et al., 2014) and in large-scale model tests (Farrag and Morvant, 2004; Benjamim et al., 2007; Won and Kim, 2007; Portelinha et al., 2013; Portelinha and Zornberg, 2017). More recently, a growing interest has emerged to better understand the behaviour of reinforced structures with cohesive soils, both by experimentation (Chen et al., 2007; Noorzad and Mirmoradi, 2010; Raisinghani and Viswanadham, 2010; Wang et al., 2011) and numerical analyses (Guler et al., 2007; Liu et al., 2009).

From the practitioner viewpoint, international standards limit the use of cohesive soils in reinforced soil structures. AASHTO (2002) limits the particles passing on sieve No. 200 in 15% and allows a maximum plasticity index of 6. Also, the minimum angle of shearing resistance should be 34°. FHWA (Berg et al., 2009) recommends soils with less than 15% of the particles passing on sieve No. 200 and a maximum plasticity index of 6. The eighth edition of AASHTO LFRD bridge design specifications (AASHTO, 2017) allows accounting for cohesion but it does not provide any detail about how to include the cohesive strength contribution in the design formulae. With regard to this, practitioners need to be acutely aware that unlike purely frictional backfills, cohesive soils are subject to the formation of tension cracks which can reduce the extra stability conferred by the cohesive contribution rather substantially as shown in Abd and Utili (2017), and for unreinforced slopes in Utili and Abd (2016), Utili (2013) and Michalowski (2013). Therefore, the formation of tension cracks need to be properly accounted for in any design relying on soil cohesion.

In this paper the design charts provided by Abd and Utili (2017) were employed in five design case studies where cohesive-frictional soils very common in Brazil are utilized as backfill. The geo-reinforced slopes are featured by different face inclinations, slope heights and soil strength parameters. Calculations of the required reinforcement strength were carried out for the two most common reinforcement distributions used in practice, i.e. uniform and linearly increasing with depth.

2. THEORETICAL FORMULATION

In Abd and Utili (2017) the structural approach was employed together with the kinematic (upper bound) method of LA to obtain lower bounds on the required level of reinforcement extending the LA formulation of Michalowski (1997) for purely frictional backfills to cohesive frictional backfills. Note that LA assumes a simplified constitutive behaviour for both ground and reinforcement, i.e. rigid – perfectly plastic, and the validity of the normality rule, i.e. associated plastic flow, which at rigor does not hold true for most soils. A comprehensive treatment of limit analysis assumptions and limitations and their implications for slope stability can be found in (Chen, 1975).

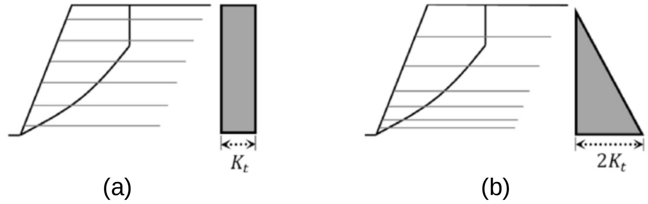
Traction-free uniform c-ϕ slopes reinforced with geosynthetic layers are considered. A common design choice for the distribution of reinforcement with depth is to employ reinforcement layers of equal strength laid at equal spacing or at a spacing decreasing linearly with depth. The former case gives rise to a uniform distribution (UD) of tensile strength over depth (see Fig. 1a) which can be expressed as:

$$K_t = \frac{nT}{H} \qquad [1]$$

with K_t being the average strength of reinforcement in the slope, n the number of reinforcement layers, T the strength of a single layer at yielding point and H the slope height. Note that the influence of the overburden stress on the strength of the geosynthetics has been neglected for sake of simplicity (Michalowski, 1997). Instead, the second case gives rise to a linearly increasing distribution (LID) of strength over depth (see Fig. 1b):

$$K = 2K_t \frac{(H - y)}{H} \qquad [2]$$

with K representing the local reinforcement strength in the slope, and y the vertical upward coordinate departing from the slope toe. Note that there is plenty of evidence from field observations and experimental tests showing the load distribution in the reinforcement for slopes under working stress conditions is non-linear (Allen and Bathurst, 2015; Yang et al., 2012) so neither a UD nor a LID. However, the assumption of UD or LID is consistent with the LA assumption of the georeinforced slope being at impending failure and of rigid – perfectly plastic behaviour for the materials of the system (ground and reinforcement) which possess infinite ductility. These two assumptions imply that the distribution of forces in the reinforcement must coincide with the distribution of reinforcement strength (Michalowski, 1997).



(a) (b)

Figure 1. Layout of the geosynthetic reinforcement: a) uniform distribution; b) linearly increasing distribution with depth.

3. EXAMPLE CASE STUDIES

The 5 case studies here considered are provided in Table 1: one large scale model reinforced slope reported in Benjamim (2006), one field reinforced slope reported in Portelinha (2014) and three hypothetical reinforced slopes with typical cohesive soils from the state of São Paulo, in Brazil. Note that in cases 1 and 2 the slope heights here adopted are different from the actual heights reported in the respective publications. In fact, a height corresponding to a normalized cohesion ($c/\gamma H$) of 0.1 was chosen for ease of calculations. The data used to design the hypothetic slopes (cases 3 to 5) were obtained from the São Paulo metro company (Companhia do Metropolitano de São Paulo, 1980) as representative of the typical soils found in São Paulo (Brazil).

Table 1. Geometry and properties of the case studies reinforced slopes

Case study	Slope Geometry		Soil characteristics						Source of data
	H (m)	β (°)	Soil type	γ (kN/m)	ϕ'_{peak} (°)	c'_{peak} (kPa)	Shear strength test		
1	30[1]	78.0	Non lateritic silty sand	20.4[2]	27	55	CD		Benjamim (2006)
2	12[1]	84.3	Lateritic silty sand	20.5[2]	29	19	CD		Portelinha (2014)
3	15[1]	80.5	Clayley sand	20.0	35	30	-		Metro SP –NC-03 (1980)
4	30[1]	80.5	Sandy clay	19.0	20	60	-		Metro SP –NC-03 (1980)
5	15[1]	80.5	Residual soil	18.0	29	30	-		Metro SP –NC-03 (1980)

[1] Slope heights imposed to result in $c/\gamma H$ = 0.10.
[2] Soil unit weight for a relative compaction of 98% of the Standard Proctor maximum density.
CD – Consolidated drained triaxial test.

In order to evaluate the potential savings in considering soil cohesion during the design of reinforced soil slopes, the design charts for null cohesion and normalized soil cohesion (c/γH) equal to 0.1 provided by Abd and Utili (2017) for the cases of UD and LID reinforcement were used. Since a fully functional drainage system is a design requirement for the use of cohesive backfills, fully drained conditions were assumed, i.e. r_u was taken equal to 0. The coefficient t expressing the amount of soil tensile strength was assumed to be equal to 0.5 in all the analyses.

In Figure 2 a flowchart illustrating all the steps required to obtain the final and most economical reinforcement option for each case study is provided. Once the value of $K_t/\gamma H$ is determined from the design charts it is possible to obtain the number of layers for a given geosynthetic tensile strength or to calculate the necessary strength of each layer (adopted as the same for all layers) if the number of layers n is prescribed. Vertical spacing was limited to a maximum of 60 cm according to the recommendations of AASHTO (2017) to avoid excessive deformation of the facing. Due to construction viability (compaction), solutions with vertical spacing less than 10 cm were not considered.

Table 2 shows the values of the required reinforcement, $K_t/\gamma H$ obtained for each case study with and without cohesion. The reductions in the required value of $K_t/\gamma H$ (relative to the non-cohesion case) were larger than 70 % for all cases, which unequivocally demonstrate the beneficial effect of accounting for the cohesive component of the backfill mechanical strength. It is worth noting that the full soil strength parameters from Table 1 were used in the analyses.

The determination of the geosynthetic tensile strength is affected by the availability of commercial materials, which implies the designer has to use a discrete range of values. The tensile strength we employed to determine the lower bound to the number of reinforcement layers is the long-term tensile strength (T_{al}), which accounts for installation damage, creep and chemical and biological degradation. This value is calculated according to Eq. [3].

$$T_{al} = \frac{T_{ult}}{RF} \qquad [3]$$

where T_{ult} is the ultimate tensile strength and RF is the combined strength reduction factor accounting for installation damage, creep and chemical and biological degradation.

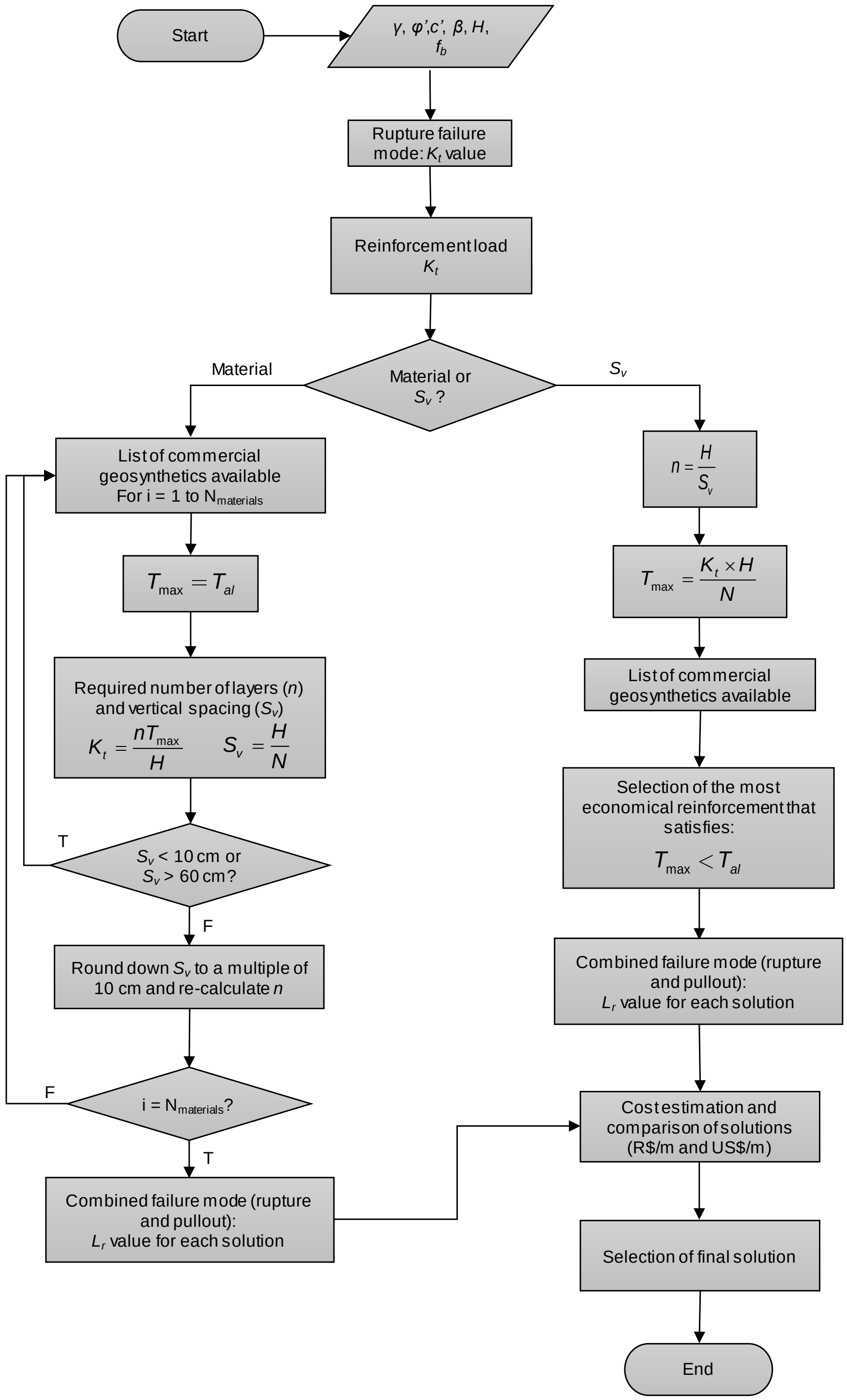


Figure 2. Steps for the design of a reinforced soil slope according to the method proposed by Abd and Utili (2017).

Table 3 shows the estimated costs for a range of materials commercially available in Brazil. The ultimate tensile strengths (T_{ult}) range from 31 kN/m to 600 kN/m. Nominal long-term reinforcement strengths (T_{al}) were obtained for each material by applying the appropriate reduction factor (Eq. [3]). Thus, materials with nominal strengths in the range of 4.5 kN/m to 328 kN/m are currently available for selection. When not provided by the manufacturer, partial reduction factors were adopted according to the recommendations of Koerner (2005). The cost estimations were based on manufacturer information and from the unit price table published quarterly by the Secretariat of Logistics and Transport of São Paulo state, in Brazil (DER, 2019). These values are average market references and were obtained from input prices surveyed by a recognized economic research institution linked to the University of São Paulo.

Table 2. Required reinforcement $K_t/\gamma H$ for each case study with no-cohesion (c = 0) and with cohesion consideration for uniform distribution (UD) and linear distribution (LID) of reinforcements

ID	No cohesion		With cohesion		
	c/γH	*Kt/γH*	*c/γH*	*Kt/γH*	Reduction in *Kt/γH*
UD					
Case 1	0	0.16	0.09	0.025	84.4%
Case 2	0	0.166	0.09	0.040	75.9%
Case 3	0	0.111	0.10	0.003	97.3%
Case 4	0	0.241	0.11	0.074	69.3%
Case 5	0	0.152	0.11	0.025	83.6%
LID					
Case 1	0	0.138	0.09	0.024	82.6%
Case 2	0	0.148	0.09	0.036	75.7%
Case 3	0	0.100	0.10	0.002	98.0%
Case 4	0	0.199	0.11	0.066	66.8%
Case 5	0	0.133	0.11	0.023	82.7%

Table 3. Normalized costs estimates of geosynthetics reinforcing elements (for 1 kN tension and for 1-m-run wall)

ID	Type	Material	T_{ult} (kN/m)	*RF*	T_{al} (kN/m)	Cost estimation (R\$/m²)	Normalized cost (R\$/kN)
GH31	NW	PP	31	6.8[1]	4.5	9.0	0.29
GH36	NW	PET	36	5.9[1]	6.2	10.5	0.29
GH40	NW	PET	42	5.9[1]	7.2	12.0	0.29
Fortrac 35T	GG	PET	35	3.3[1]	10.8	21.7	0.62
Fortrac J700 MP	GG	PVA	35	3.3[1]	10.8	21.7	0.62
GH50	NW	PET	70	5.9[1]	12.0	13.5	0.19
Fortrac 55T	GG	PET	55	3.3[1]	16.9	24.9	0.45
Fortrac J1100 MP	GG	PVA	55	3.3	16.9	24.9	0.45
WG40	GG	PET	40	1.8	21.9	21.7	0.54
WG40S	GG	PET	40	1.8	21.9	21.7	0.54
WG50S	GG	PET	50	1.8	27.4	24.9	0.50
WG60	GG	PET	60	1.8	32.8	24.9	0.41
WG65S	GG	PET	65	1.8	35.6	24.9	0.38
WG90	GG	PET	90	1.8	49.3	28.0	0.31
WG120	GG	PET	120	1.8	65.7	31.2	0.26
WG150	GG	PET	150	1.8	82.1	35.9	0.24
WG200	GG	PET	200	1.8	109.5	39.8	0.20
WG200S	GG	PET	200	1.8	109.5	39.8	0.20
WG300	GG	PET	300	1.8	164.2	47.7	0.16
WG400	GG	PET	400	1.8	218.9	47.7	0.12
WG500	GG	PET	500	1.8	273.7	39.6	0.08
WG600	GG	PET	600	1.8	328.4	23.5	0.04

The normalized cost shown in Table 3 allows comparing the relative cost per unit of tensile strength of the materials which can help in selecting the reinforcement material with the best cost-performance efficiency.

Table 4 and Table 5 show the solutions obtained with consideration of the rupture failure mode (prior to the verification of pullout) and for a uniform distribution (UD) of tensile strength over depth. The onset of tension cracks was considered in the analyses with the maximum crack depth being constrained according to Abd and Utili (2017). T_{req} is the required tensile strength calculated from Eq. [1]. The material properties of the reinforcement were selected from the list of geosynthetic products commercially available in Brazil (see Table 3) so it would give $T_{al} \geq T_{req}$. The best solutions are obtained by the choice of larger vertical spacings and larger reinforcement tensile strengths, in both analyses (without and with soil cohesion). However, even though the number of layers for each study case does not change by considering soil cohesion, much lower tensile strengths are required, which alters the choice of material. This has a direct impact on the cost estimation, with reductions from 20% up to 58%.

The results presented in Table 4 and Table 5 show the potential that the consideration of cohesion in design has on the financial costs of a reinforced soil slope. However, the final solution can only be obtained after the determination of the required length of reinforcement, which depends on the number of layers selected. A decrease in the number of reinforcement layers (consequently the vertical spacing) causes an increase in the minimum required reinforcement length, which can disfavor a solution with larger vertical spacing and stronger reinforcement materials, since the final cost per wall wide (R$/m or US$/m) is a function of the reinforcement length. To illustrate this fact, the required reinforcement lengths were calculated according to the procedure reported in Abd and Utili (2017) for Case 3 and Case 5 for two vertical spacings (30 cm and 60 cm). The results are presented in Table 4 and Table 5, without and with consideration of soil cohesion, respectively. The two cases share the same slope height and inclination but different soil parameters are employed (Table 1).The bond coefficient between soil and reinforcement (f_b) was taken equal to 0.6. Changing the vertical spacing from 60 cm to 30 cm bears a negligible effect on the required lengths for both cases considered. Therefore the adoption of a larger spacing results in smaller costs. By comparing the analyses performed with and without cohesion, the reduction in L/H turns out to be rather large, producing savings in the order of 70% on the cost of reinforcement.

Table 4. Solution and cost estimation (R$/m²) considering no cohesion, for a uniform distribution of reinforcement (UD).

Case	H (m)	S_v (cm)	n	ID Material	T_{al}/T_{req}	Cost (R$/m²)	Cost [1] (US$/m²)	L/H	Cost (R$/m)	Cost [1] (US$/m)
Case 1	30	60	50	WG120	1.12	1557.5	389.4	-	-	-
Case 2	10	50	20	WG40	1.29	434.2	108.6	-	-	-
Case 3	15	30	50	Fortrac 35T	1.08	1085.5	271.4	0.57	9346.5	2336.6
Case 3	15	60	25	WG40	1.10	542.8	135.7	0.58	4729.0	1182.3
Case 4	30	60	50	WG200	1.33	1991.5	497.9	-	-	-
Case 5	15	30	50	Fortrac 55T	1.37	1242.5	310.6	0.75	13921.8	3480.5
Case 5	15	60	25	WG50S	1.11	621.3	155.3	0.76	7055.6	1763.9

[1]Adopted USD exchange rate at 26/10/2019: 1 R$ = 0.25 US$

Table 5. Solution and cost estimation (R$/m²) considering soil cohesion, for a uniform distribution of reinforcement (UD).

Case	H (m)	S_v (cm)	n	ID Material	T_{al}/T_{req}	Cost (R$/m²)	Cost[1] (US$/m²)	L/H	Cost (R$/m)	Cost[1] (US$/m)
Case 1	30	60	50	Fortrac 55T	1.37	1242.5	310.6	-	-	-
Case 2	10	50	20	GH36	1.11	210.0	52.5	-	-	-
Case 3	15	30	50	GH31	12.46	450.0	112.5	0.37	2498.4	624.6
Case 3	15	60	25	GH31	6.23	225.0	56.3	0.37	1250.7	312.7
Case 4	30	60	50	WG65S	1.04	1242.5	310.6	-	-	-
Case 5	15	30	50	GH31	1.66	450.0	112.5	0.50	3382.7	845.7
Case 5	15	60	25	GH36	1.13	262.5	65.6	0.50	1986.3	496.6

[1]Adopted USD exchange rate at 26/10/2019: 1 R$ = 0.25 US$

It is worth mentioning that the present study dealt only with the internal stability design of the reinforced soil slopes case studies and no load or resistance factors were applied. The complete design of such structures should be conducted with consideration to external stability verifications such as foundation bearing, sliding and overturning.

4. METHOD VALIDATION

Validation of a new design method can be performed either by numerical or by experimental analyses. In Abd and Utili (2017), the design method proposed and here utilized was validated by using finite element displacement-based analyses with strength reduction technique (FESR) and finite element limit analyses (FELA) showing that the amount of reinforcement required from the analytical kinematic approach (lower bound) was slightly greater than the FELA lower bounds and was less than 14% smaller than the value obtained from FELA static approach (upper bound). Since the true collapse values lie in between the lower and upper bounds, the authors concluded that the required reinforcement obtained from the new method is adequate to be used in the proposed design charts.

Experimental validation would require data on full-scale reinforced slopes brought to failure, which are not currently available for the case of cohesive backfills. An alternative here pursued is to use published studies of reinforced slopes brought to collapse in the geotechnical centrifuge which are available for cohesive backfills. Cases 6 to 9 in Table 6 report tests on four reduced-scale reinforced model slopes brought to failure in the geotechnical centrifuge by Porbaha and Goodings (1996, 1997). From Springman et al. (1992), Porbaha and Goodings (1997) and Viswanadham and König (2004 (2004), the relevant scale factors for the present study can be calculated as:

$$\frac{S_m}{S_p} = \frac{H_m}{H_p} = \frac{T_{ult,m}}{T_{ult,p}} = \frac{1}{N_f} \qquad [4]$$

where S is the reinforcement spacing, H is the slope height, T_{ult} is the ultimate reinforcement tensile strength and N_f is the gravitational acceleration at failure. Subscripts m and p refer to the model and prototype, respectively.

Table 7 shows further details of the model slopes and the equivalent prototype data. The same non-woven polyester geotextile with uniform spacing (8 layers) was used in all model slopes. For the cases presented, the study variables were restricted to the reinforcement length/slope height ratio (L/H varied from 0.5 to 0.75) and slope face inclination (71.6° and 80.5°). Porbaha and Goodings (1996) observed that the geotextile strain zone in the models was limited to a much narrower distance than the gauge length used in the wide-width tensile tests specified in ASTM standard test (ASTM D4595, 2017). The 'zero-span' tensile strengths presented in Table 7 are related to tensile tests conducted with a gauge length of 6 mm, which would be more representative of the geotextile strength in the conditions of the model slope test.

Figure 3 presents the comparison of the tensile strength required according to the approach proposed by Abd and Utili (2017). The reinforcement required tensile strength T_{req} is 2 to 5 times greater than the wide-width tensile strength of the materials used in the models and up to 2.1 times greater when compared with the values from the zero-span tests. The closer values in the latter case were expected since a narrow strain distance was observed to occur in the reinforced soil models (Porbaha and Goodings (1996)). A slightly larger difference was observed for Case 9, the steeper slope, with the T_{req} in the order of 5 and 2.1 times the wide-width and the zero-span tensile strength, respectively. Therefore, it can be concluded that the method proposed in Abd and Utili (2017) was able to calculate the reinforcement strength required for the design case studies here considered, giving a minimum required tensile strength larger than the material strength correspondent to the failure of the models, especially if compared with material strengths values determined via the standard wide-width tensile test.

Table 6. Geometry and properties of the case studies reinforced models (after Porbaha and Goodings (1996, 1997))

Case study	Slope Geometry		Soil characteristics				Shear strength test	Source of data
	H (m)	β (°)	Soil type	γ (kN/m)	ϕ'_{peak} (°)	c'_{peak} (kPa)		
6	13.1	71.6	Hydrite kaolin	17.8	19.5	23.3	DS	Porbaha and Goodings (1996)
7	12.6	71.6	Hydrite kaolin	17.8	18.3	21.4	DS	Porbaha and Goodings (1996)
8	10.2	71.6	Hydrite kaolin	17.8	20.6	16.4	DS	Porbaha and Goodings (1996)
9	11.1	80.5	Hydrite kaolin	17.8	21.3	22.7	DS	Porbaha and Goodings (1997)

DS – Direct shear test (retrieved samples)

Table 7. Model and equivalent prototype data at slope failure for the case studies from 6 to 9 (after Porbaha and Goodings (1996, 1997))

Case study		6	7	8	9
ID on the source study[1]		M43	M44	M47	M35
L/*H*		0.5	0.67	0.75	0.75
β (°)		71.6	71.6	71.6	80.5
$S_{v,m}$ (m)		0.019			
H_m (m)		0.152			
$T_{ult,m}$ (kN/m)	Wide-width	0.053			
	Zero-span	0.117			
N_f		67.1	82.9	67.1	73.0
$S_{v,p}$ (m)		1.3	1.6	1.3	1.4
H_p (m)		10.2	12.6	10.2	11.1
$T_{ult,p}$ (kN/m)	Wide-width	3.56	4.39	3.56	3.87
	Zero-span	7.85	9.70	7.85	8.54

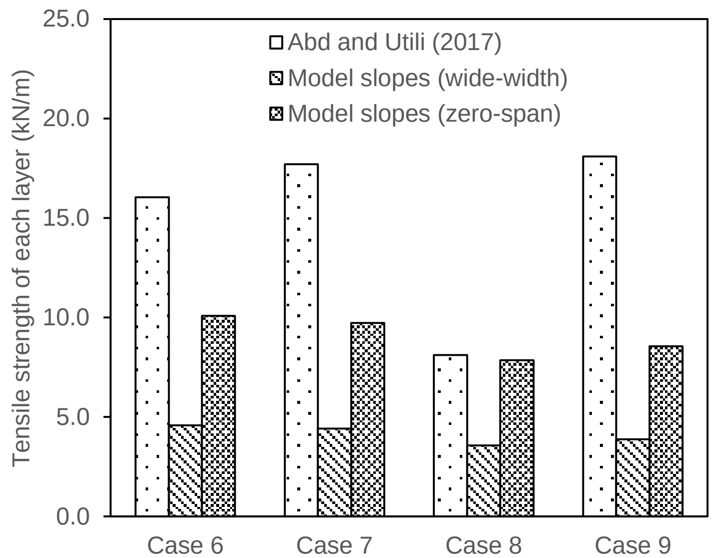


Figure 3. Comparison of the required reinforcement tensile strength (not factored) with the tensile strength of the geotextiles used in the slope models

5. CONCLUSIONS

The main focus of this paper is to showcase the potential of the the methodology proposed by Abd and Utili (2017) to achieve a substantially more economic design of reinforced slopes in cohesive-frictional backfills by correctly accounting for the beneficial contribution to soil strength of cohesion. To this end, several case studies were considered for typical cohesive soils from Brazil. A significant reduction in the amount of reinforcement required for the safe design of the slopes considered was achieved that in turn translated into important reduction of overall costs.

Data from published studies on reinforced slope models brought to failure in the geotechnical centrifuge were used to validate the method experimentally.

6. ACKNOWLEDGEMENTS

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