Design of Geosynthetic Reinforcement for Embankments on Soft Soil Considering the Strength Increase of Foundation Soil due to Consolidation

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ABSTRACT: Geosynthetics as reinforcement of embankments on soft soil are generally designed considering the end of construction solicitation and the final instant of the reinforcement service, taking into account the strength decrease due to creep. This loss is gradual in time; also, the resistance of the foundation soil changes in time due to the consolidation process. This paper discusses some of the considerations that one could adopt to analyse the stability of a reinforced embankment on soft soil, by considering time as a design parameter. It is presented, in order to illustrate, some results of an idealised procedure of analysis to take these aspects into account. Limit Equilibrium analysis are performed and compared to Finite Element results. Terzaghi's consolidation theory is the basis for the consolidation analysis; some hypothesis and limitations are discussed.

1 INTRODUCTION

Reinforcement of embankments on soft soil is an important contribution of the geosynthetics to Geotechnical Engineering being largely applied worldwide.

The design of geosynthetics reinforcement for embankments on soft soil is traditionally performed by the analysis of stability conditions of the structure through methods that consider limit equilibrium principles. In general, analyses via such methods require some input parameters; two of them are subject of the paper: the soft soil undrained shear strength and the reinforcement available strength.

In most cases, the foundation soft soil is some kind of fine and saturated soil. Hence, its shear strength is normally estimated from “in situ” tests, and it is taken as undrained strength. This is a critical parameter, once stability condition is very sensitive to its magnitude.

In the case of the geosynthetic reinforcement, in such application, tensile strength and stiffness are the most important parameters. During the service lifetime, these parameters are significantly affected by creep behaviour of the reinforcement material.

In design time, these parameters are normally considered as following: the soil shear strength is estimated according to the initial condition (before the embankment building), and the reinforcement available strength is estimated for a predetermined reinforcement service time (usually taken as the time lapse for 90% of soil consolidation). In an opposite point of view, some people believe that the reinforcement strength decrease (creep) is not an issue for embankments on soft soils, once soil should gain strength through consolidation faster than geosynthetics looses it. The first alternative is recognised as conservative, once parameters are taken at its critical values; the second is not a correct idea, once creep is always very important for polymeric materials, even for short-term loads.

This paper discusses analysis criteria to take account for changes on these parameters through time. In general, hypotheses and concepts from Terzaghi’s theory are adopted for soil consolidation process; on the processes of changes in geosynthetics available strength, behaviour of polymeric reinforcement materials in creep are considered. Moreover, some limitations of these hypotheses are discussed with some emphasis in the way they affect this work’s proposals and the Limit Equilibrium Analyses. Firstly, it is presented an overview about the two subjects: saturated soils consolidation and creep of geosynthetics.

The graphs presented in Figure 1 illustrate qualitatively the essence of the proposal of the work.

Hence, the aim of this work is essentially to discuss analyses procedures, considering the influence of time over soil resistance and reinforcement tensile strength.

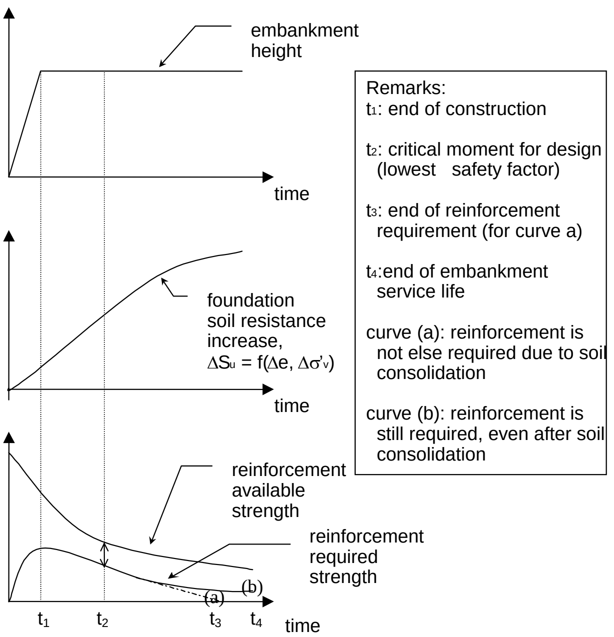

Figure 1. Qualitative illustration of the proposal.

2 MATERIALS CHARACTERISTICS AND BEHAVIOUR

2.1 *Geosynthetics*

2.1.1 *Geosynthetic Reinforcement Design Strength*

Available Strength, or Long-Term Design Strength (LTDS), is the strength parameter of the synthetic reinforcement that should be used in project analysis. However, product catalogues often provides only the Ultimate Tensile Strength (UTS) (ISO 10319

1993), which is a nominal value from a short-term tensile test. UTS should be considered only as a reference.

In order to estimate the design strength from these nominal values, reduction factors, which are also called (sometimes improperly) safety factors, must be applied. These reduction factors accounts for reinforcement strength losses, due to site and load conditions during service lifetime.

Main factors that contribute to strength decrease are:

- Creep;
- Mechanical damage (during installation mainly): dependent on site conditions and products physical characteristics;
- Environmental damage (hydrolysis, UV, chemical exposition): dependent on soil conditions (particles and chemical composition), product polymer and eventually time.

Creep and environmental effects are time-dependent, but the former won't be focused here. Hence, among the other reduction factors, the most important one is related to creep phenomenon, due to its magnitude, which is usually much higher than the magnitude of the other factors.

The total reduction factors of the mentioned items, in general, may lead to a great discount to the reinforcement tensile strength value. Generic suggestions for such reduction factors may be obtained from literature, for each type of geosynthetics and polymers. Certified product suppliers, otherwise, are allowed to provide specific reduction factors for their products, which could be less conservative.

So, the estimation of available strength (LTDS) may be done through (Koerner, 1998, Jewell, 1996):

$$LTDS = \frac{UTS}{f_{cr} \times f_{md} \times f_{env} \times f} \qquad (1)$$

where: f_{cr} is the reduction factor due to creep; f_{md} is the reduction factor due to mechanical damage; f_{env} is the reduction factor due to environmental damage; f is the safety factor due to eventual uncertainties

2.1.2 *Creep*

Creep phenomenon is the long-term deformation of a material when exposed to a permanent load. Geosynthetics, as they are polymeric materials, are very susceptible to such phenomenon. The magnitude of the creep influence on strength loss depends upon the loading time and upon the load itself. Creep is also strongly affected by the environment temperature.

Another important aspect should be mentioned: in general, creep phenomenon is more intense in the very first instants of a long-term solicitation, for all polymer types.

Figure 2 presents for instance, the isochronous curves for a certified woven geogrid, made of high tenacity polyester, provided by the material supplier.

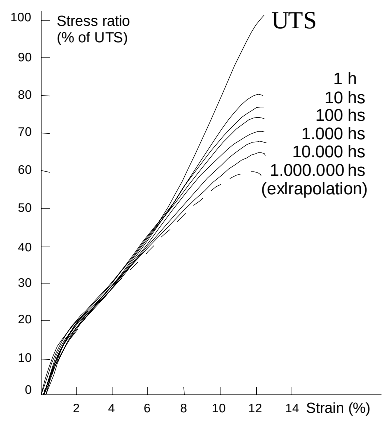

Figure 2. Example of isochronous curves for a geogrid tensile strength (for all Fortrac geogrids between 0 and 30^{o}C, BBA Certificate, 1999).

2.2 *Foundation Soft Soil*

The role of consolidation over stability of embankments is well known among geotechnical engineers: as consolidation evolves, effective stresses grow up and both soil stiffness and shear strength increase. In spite of the existence of constitutive models that take these phenomena into account, simpler models have been systematically preferred in the design of embankments.

The simplest constitutive model used by geotechnical engineers is the rigid-plastic model, that is used in limit analysis. This model is largely used in limit state analysis, in which soil and reinforcement stiffness are neglected, and only strength is taken into account.

Linear elastic-perfectly plastic models are often used in finite elements analyses. This kind of model can take into account the relationship between initial confining stress and (initial) stiffness.

Regarding to soil strength, both rigid-plastic and elastic perfectly plastic models may be used either in undrained condition (assuming friction angle equals to zero), or with Mohr-Coulomb failure criterion. In the first case, the model won't be able to take naturally into account strength dependence to confining pressure, unless to the extent of initial conditions. In the second case, the model will take into account the effect of confining tension over soil strength.

Sophisticated constitutive models like Janbu's Hyperbolic model and Critical State models (Wood, 1990) can take into account that confining stresses can change both soil stiffness and strength. Although Critical State models can also take into account pre-consolidation effects, the main reason for the choice of these models in this work is based on its large past use to model Brazilian soft clays (Lacerda & Almeida, 1995, Almeida, 1982), from which reliable parameters could be obtained.

In general, the soft soil consolidation is analysed by Terzaghi's Theory (Lambe & Whitman, 1979), established for one-dimensional soil deformations and water flow. In an embankment over soft soil foundation problem, these hypotheses could be valid only in a region below the embankment, far enough from the slope edge. Once an embankment is not an infinite uniform load, velocity of consolidation is underestimated due to different mechanisms of water flow and vertical stress increments.

3 ADOPTED EXAMPLE

3.1 *Geometry*

To exemplify this work's discussion, an hypothetical problem is posed by an embankment 4m high with a 2h:1v slope, to be constructed over a 10m very soft soil foundation layer. It was adopted the groundwater level to be coincident with the soft soil surface, with drainage through the top and the bottom of the ground.

3.2 *Soft Soil*

3.2.1 *Limit Equilibrium Analyses*

The soft soil properties adopted for the example was taken from literature (Lacerda & Almeida, 1995, Almeida, 1982), based on a typical foundation soil from Sarapuí county, in state of Rio de Janeiro, Brazil. This choice is due to a large number of studies carried over this soil by many researchers from COPPE/UFRJ. Table 1 presents the most relevant parameters for the analyses.

To take into account the time effect over the undrained shear strength of the soft soil, it was taken as valid the relationship between undrained strength and maximum effective vertical stress, given by (Lacerda & Almeida, 1995):

$$S_u = 0.35\ \sigma'_{vm} \qquad (2)$$

Table 1. Soil parameters of Sarapuí clayey soil.

Parameter	Unit	Mean Value
Total depth	m	10*
% clay	%	50
% organic material	%	5.25
Wet unit weight: γ	kN/m^3	13-14
Preconsolidation stress	kN/m^2	15**
Compression ratio: $C_c/(1+e_0)$	-	0.38
C_r/C_c	-	0.12
Undrained strength: $S_{u\ (vane)}$	kN/m^2	8 – 15***
S_u/σ'_{vm}	-	0.35
Consolidation coefficient: C_v	m^2/s	2.0 E-8**

* 10m was considered for the analysis, different from the 11m published by Lacerda & Almeida, 1995.
** values chosen in the range published by Lacerda & Almeida, 1995.
*** 8 kN/m^2 down to depth of 3 m, increasing downwards according to σ'_v (see equation 2).

3.2.2 *Finite Element Analysis*

The Finite Element code Plaxis (Brinkgrere & Verneer, 1998) has a Critical State model very similar to MCC (Modified Cam-Clay, see Wood, 1990), called Soft Soil Model (SSM). This model has some "extra" properties, like cohesion to the critical state line. Although this characteristic may find some utility in numerical stability, some care should be taken in the use of this parameter, as it could change drastically the model behaviour at low ratios between confining tension and this "cohesion". Following Plaxis' advise, a small nominal cohesion was adopted, in order to enhance numerical convergence of the analysis. Table 2 presents the most relevant complementary parameters for the analysis.

Table2. Complementary soft soil parameters for the FEM analysis

Parameter	Unit	Value
Permeability (K_x=K_y)	m/day	4.3E-4
Initial void ratio	-	2.7
Cohesion (SSM)	kN/m^2	1
Friction angle (SSM)	º	28
λ (SSM)		0.165
κ (SSM)		0.066
Deformability E_{ref} *	kN/m^2	700
ν_{ur}		0.35
POP (SSM)	kN/m^2	27

* 3000 kN/m^2 until 3m deep

3.3 *Embankment*

For the hypothetical embankment, one typical clayey sand was chosen:

- Wet unit weight: γ = 20kN/m^3
- Effective cohesion: c' = 5kN/m^2
- Effective friction angle: ϕ' = 30°

3.4 *Reinforcement*

The relationship between time and tensile strength was interpolated from peaks in isochronous curves (Figure 2). These curves were also used to estimate the reinforcement stiffness to be used in Finite Element analysis (Figure 3), which was based on a strain of 5%. The stiffness at this level of strain represents well the behaviour of the adopted geogrid in the strain range from 4 to 8 %, and is slightly conservative for smaller strains. In Equilibrium Analyses, tensile strength was taken as time dependent by using rupture strength in Ultimate Analysis and 5% strain for Serviceability Limit State Analysis (Figure 3).

Through these curves it was possible to take out the creep behaviour of the chosen geosynthetic, as well as the creep reduction factors valid for it. Other reduction factors are considered herein (for analytical purposes only) as 1.0, as they are not subject of the present discussion.

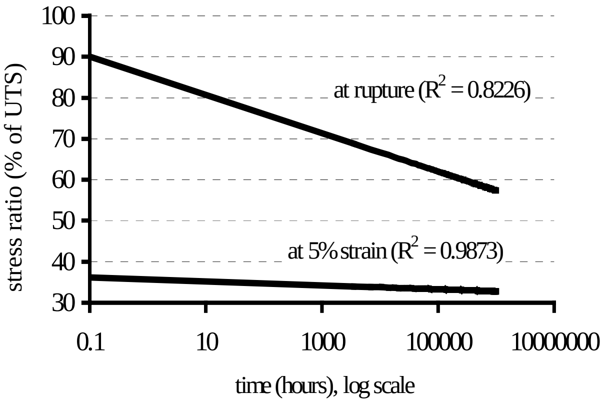

Figure 3. Reinforcement tensile strength time depending curves.

4 SOIL CONSOLIDATION ANALYSIS

Figure 4 presents the increasing of the undrained shear strength due to soil consolidation by Terzaghi's Theory for an equivalent load of an embankment 4m high. Figure 5 presents the velocity of consolidation obtained from the same analysis.

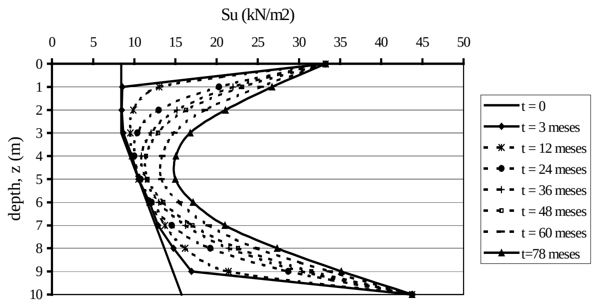

Figure 4. Shear strength due to soil consolidation for the example.

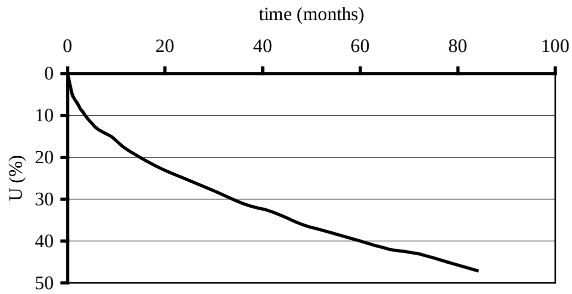

Figure 5. Percentage of consolidation for the adopted example.

Terzaghi's consolidation analysis (Pinto, 2000) indicated a total settlement of 2.04m and 90% of consolidation occurring in an approximate time lapse of 409 months.

5 LIMIT EQUILIBRIUM ANALYSES

The analyses were performed on software suitable for soil structures stability analysis, GGU-Slope (Buβ, 1999). This is a German commercial package that calculates limit equilibrium analysis by five different methods, allowing the inclusion of geosynthetic reinforcements. It was used the Simplified Bishop Method, that consider a circular failure surface (Lambe &

Whitman, 1979).

The information about reinforcement material required by GGU-Slope are a design tensile strength value and a parameter of reinforcement/soil interaction, like an interaction coefficient. The soft soil is treated as a Mohr-Coulomb material ($c=S_u$ and $\phi=0$) and it may be horizontally stratified in order to simulate a variation of the soil resistance with depth.

The analyses were performed for time lapses of 0, 1, 2, 3, 6, 9, 12, 18, 24, 30, 36, 42, 48, 54, 60, 66, 72 and 78 months after embankment construction; after that, no reinforcement was required by Limit Equilibrium Analysis. Soft soil strength in each of these instants was calculated by Equation 2, where effective vertical stress (σ'_{vm}) was obtained by Terzaghi's Theory (Lambe & Whitman, 1979). Under the embankment plateau, a load equivalent to a 4m high embankment was adopted, while under the slope region, a 2m high embankment was considered. This last assumption is a common practice, although a greater refinement of slope region might lead to better approximation.

The results of the analyses are presented at Figures 6 and 7. Stability Analysis were performed following the concepts of Ultimate Limit States, but safety factors of 1.0 were adopted, in order to simplify the discussions and comparisons.

The results are presented in terms of the desired Ultimate Tensile Strength (UTS) of the polymeric material. The analyses were performed, in other hand, in terms of Long Term Design Strength (LTDS). The UTS value was obtained from the LTDS achieved in each case (Equation 1). All considerations of available strength were done based on the curves of Figure 3.

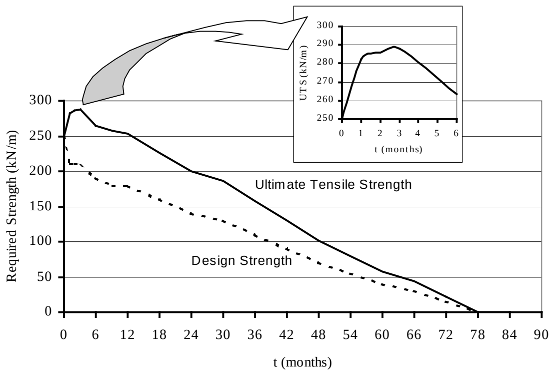

Figure 6. Ultimate Limit State analyses for safety factors = 1.0.

From these analyses it was possible to obtain the curves presented on Figure 6 (see detail), which allows determination of the critical service instant for the structure, when the strongest reinforcement would be required.

For the case analysed, the peak value was achieved at the third month of structure service. Herein, the period of construction was not taken into account, to simplify the analysis. In other words, the initial time of the analysis is the end of an "instantaneous" embankment construction.

It is worth noting that the most critical moment is neither necessarily the beginning of the structure lifetime nor the reinforcement service period. It mainly depends on both foundation resistance increase and reinforcement available strength decrease, and on velocities of both processes.

Figure 7 presents the maximum depth of critical failure surface (D), achieved by stability analyses, as function of time. It is noticeable that this value tends to decrease as the soft soil gains strength and geogrid loses it. Exception should be made to the very first instant, (time zero), were no soil strength increase has occurred yet, and undrained shear strength has a different profile when compared to other instants. After the beginning of soil consolidation, significant shear strength increase occurred on the upper and on the deeper horizontal layers of the foundation soil. However, as many phenomena occur simultaneously, like soil resistance increase and reinforcement strength decrease, it is not possible to state any general conclusion based on only one simulation.

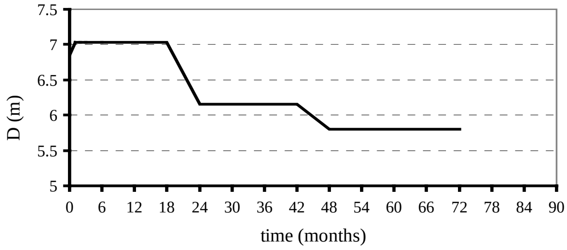

Figure 7 Depth of the most critical failure surface.

Table 3 presents the results of stability analyses, carried under two principles:

– the one presented here, called Idealized Procedure, where the foundation soil strength increase is taken into account. In order to allow comparison, it was taken the most critical instant along the whole structure service life (the 3rd month for ULS analysis);
– the one that is here called Traditional Procedure, where no soil resistance increase is considered, and the reinforcement tensile strength is estimated to the time lapse of 90% soil consolidation (Jewell, 1996) and to the end of its service life (78 months, in this case).

Table 3. Design results (for safety factors = 1.0).

	Idealized Procedure	Traditional Procedure	
	(A)	(B)	(C)
UTS (kN/m)	288	365	376
Comparison	100%	127%	131%

(A) for 3 months of reinforcement service
(B) for 78 months of reinforcement service
(C) for 409 months, 90% of soil consolidation

By comparing the results obtained above, it is clear that the Traditional Procedure is much more conservative than the Idealised one. By the results from this example, UTS obtained by Traditional Procedure is about 27% or 31% greater then UTS calculated by accounting for time changes on the parameters.

It is worth remarking that the difference between the values obtained through Traditional Procedures for 78 or 409 months are less significant than their differences to the value obtained through what is called here Idealised Procedure. It may be explained by the fact that creep is much more intensive in the initial instants of reinforcement loading.

The creep reduction factor for the adopted geogrid to a loading period of 3 months is 1.37, for 78 months, 1.46 and for 409 months, 1.50. So, creep reduction factor should be always considered, even for short-term loads.

All the analyses previously presented were based on partial and global safety factors equal to 1.0. Table 4 presents results for the same studied case, considering partial safety factors according to BS 8006 (1995) recommendations for design. Ultimate Limit State (ULS) and Serviceability Limit State (SLS) analysis were performed.

For the SLS analysis it was considered a maximum allowable reinforcement strain of 5% (see section 3.4). In this case, the critical instant in the SLS analysis was the initial moment, just by the time of the end of embankment construction (here considered as instantaneous, to simplify the analysis). It was noticed that the critical moment according to each concept, ULS and SLS, are not necessarily the same. Also, the critical moment is not necessarily the beginning or the end of the structure lifetime or the reinforcement service period. Once again: it mainly depends on both foundation strength increase and reinforcement available strength decrease processes.

Table 4. Design results (for safety factors according to BS 8006, 1995).

	Idealized Procedure		Traditional Procedures (B)		Traditional Procedures (C)	
	ULS	SLS	ULS	SLS	ULS	SLS
UTS (kN/m)	600	695	947	781	978	790
Decisive UTS (kN/m)	695 (SLS)		947 (ULS)		978 (ULS)	
Comparison	100%		136%		140%	

(B) for 78 months of reinforcement service
(C) for 409 months, 90% of soil consolidation

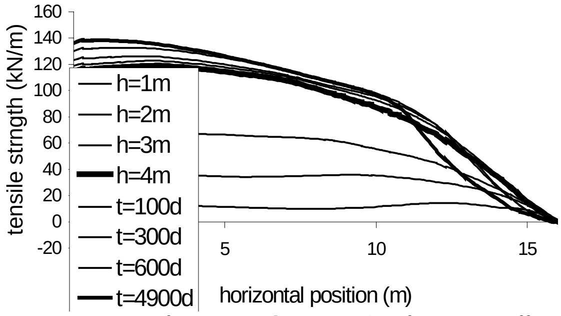

Figure 10. SSM reinforcement solicitations (reinforcement stiffness corresponding to a 500kN/m UTS product).

6 FINITE ELEMENT ANALYSIS

Finite Element analyses were performed with the code Plaxis, considering for the foundation soil a Mohr-Coulomb undrained model and a Critical State model called Soft Soil Model (SSM). Figure 8 presents a deformed mesh result to illustrate the geometry conditions. An incremental analysis was performed, in order to account for the stiffness of the reinforcement decreasing with time, according to Figure 3.

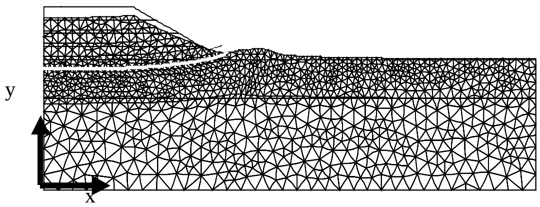

Figure 8. Example of a deformed mesh.

Mohr-Coulomb model may be treated under two concepts:

- unconsolidated undrained mode, working only with the cohesion parameter (ϕ=0), where no automatic soil strength increasing would be possible;
- consolidated undrained mode ($\phi \neq 0$), which considers strength increasing with confining stress.

The Critical State model is considered a better tool to analyse the soft soil behaviour. Figures 9 and 10 allow to compare some results obtained by MC and SSM models. The SSM analysis presents more confident and critical results. So, it was the one chosen for the discussion.

As the time lapse for construction implies an increase on foundation strength and a decrease on reinforcement stiffness, it was assumed, for the example, a consolidation time of 4 days for each 1m height of constructed embankment.

Figure 11 illustrates the total displacements obtained at the end of construction, for SSM analysis with reinforcement stiffness corresponding to UTS=700kN/m. Figures 12 and 13 present some results for the pore pressure excess on this case.

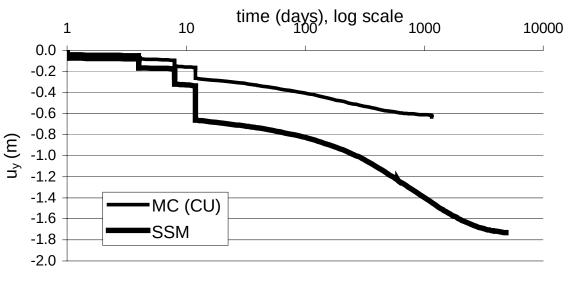

Figure 9.Vertical displacement, values at x=0 obtained by FE analysis (reinforcement stiffness corresponding to a 500kN/m UTS product).

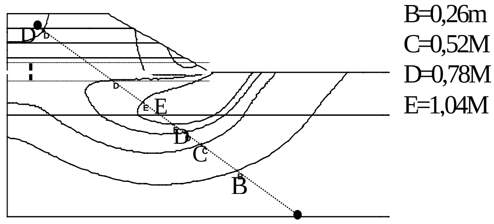

Figure 11. Total displacements for SSM analysis (reinforcement rigidity corresponding to a 700kN/m UTS product).

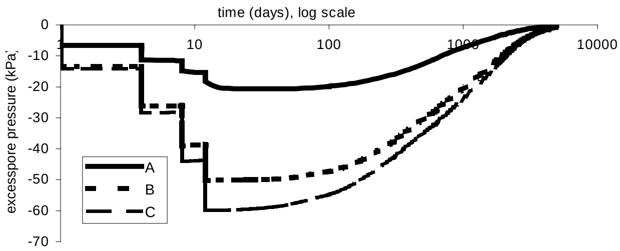

Figure 12. Variation of the pore pressure excess in time at 5m deep: A (x=15m), B (x=6m), C (x=0,5m) for SSM analysis with a 700kN/m UTS reinforcement.

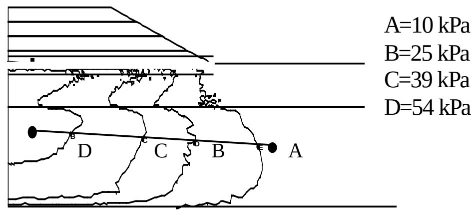

Figure 13. Excess of pore pressure at the end of construction for a SSM analysis with a 700kN/m UTS reinforcement (extreme value - 64kPa).

Table 5 presents some results obtained for various reinforcement UTS conditions, at the end of construction and at the end of consolidation. These results show the importance of the reinforcement stiffness to reduce interface displacements.

Table 5. Effect of the reinforcement stiffness.

UTS (kN/m)	End of construction ρ* (m)	End of construction u_h** (m)	End of construction Fm*** (kN/m)	End of consolidation ρ* (m)	End of consolidation u_h** (m)	End of consolidation Fm*** (kN/m)
300	0,98	0,61	109	2,14	0,66	122
400	0,90	0,48	113	2,05	0,52	131
500	0,85	0,41	119	2,00	0,43	139
700	0,80	0,31	128	1,94	0,33	151

* Maximum vertical displacement
** Maximum horizontal reinforcement displacement
*** Maximum reinforcement force

The Finite Element analysis considering a SSM (Soft Soil Model) indicates a consolidation time of 163 months.

7 ANALYSIS OF THE RESULTS

A comparison between consolidation analysis by Terzaghi's theory and results presented in Figures 12 and 13 indicates clearly that the former isn't the most adequate tool to estimate the increase of soft soil strength to be used in Limit Equilibrium Analysis. This is an expected result: one of the Terzaghi's hypotheses is that the load (embankment) should be horizontally unlimited and no horizontal flow would occur. Once the embankment is a finite load, significant horizontal water flow occur and the consolidation is much faster in reality. On the other side, stress distribution over the foundation soil causes the final increment on vertical stress to be lower than that predicted by Terzaghi's theory, as well as the final strength increment. Nevertheless, it is worth noting that for a region far enough from the embankment slope toe, results obtained by FEM analysis are very similar to the Terzaghi's ones, in this work's example. Besides that, the results obtained from Traditional Procedure will be in the safe side (see section 4), provided that failure surface does not get too much deep in foundation soil.

Figure 14 shows a comparison between:

- required strength obtained from an Ultimate Limit State Analysis through Limit Equilibrium concepts, accounting for increase in soft soil undrained strength;
- the reinforcement available strength for the 700kN/m UTS at 5% strain high tenacity polyester product designed according to the so called Idealised Procedure;
- the reinforcement solicitations calculated by Finite Element analysis considering a Critical State model and reinforcement stiffness at 5% strain (function of time) for a product with UTS = 700kN/m.

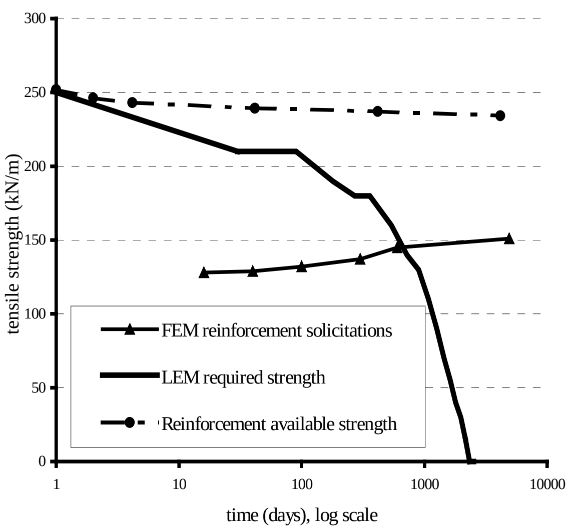

Figure 14. Comparison between some results.

It is worth noting from FEM results that the consolidation process implies continuous displacement of soil/reinforcement interface. This fact should explain the monotonic increase in reinforcement solicitation, once reinforcement is led to work under crescent levels of strain. Even after the reinforcement isn't necessary anymore to ensure the structure stability (due to soil resistance increasing), it still works restraining horizontal displacements, mainly.

Another issue: it is important to remind that the creep susceptibility of reinforcement geosynthetics depends on the nature of its raw material (polymer and other components), manufacture processes and the environment temperature. Other products could present a greater decrease in available strength values. This could lead to very different critical instant for stability analysis and, eventually, it could establish a second instant to be analysed in terms of reinforcement solicitation as obtained from FEM analysis.

8 CONCLUSIONS

From the analysis performed and the results presented (for a chosen example), some conclusions may be undertaken:

- About the consolidation process: Terzaghi's Theory underestimated consolidation time for the example analysed, but it is on the safe side and it is a good tool on the collection of parameters to be input in stability analysis considering soil consolidation with consequent resistance increase;
- About the reinforcement creep: creep is very significant for polymeric products, mainly on the initial moments of loading; so, it should be always considered on reinforced embankments stability analyses;
- About general structure behaviour: even after consolidation (and consequent soil strength increase) has developed enough to ensure the embankment stability, reinforcement plays still an important role, by ensuring serviceability conditions to the structure; on this issue, the stiffness of the geosynthetic reinforcement is the parameter to be analysed and specified;
- About the stability analysis procedure suggested (idealized): there is no doubt that it is a less conservative analysis when compared to traditional ones, once it really takes the soil resistance increase into account and do not penalise in excess the reinforcement strength value in terms of decreasing due to creep susceptibility.

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