Design of reinforced embankments: limit equilibrium and numerical methods

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ABSTRACT: The aim of this paper is a comparative study of a reinforced embankment using two different design approaches. The first methodology is based on the limit equilibrium approach and the fundamental aspect of achieving equilibrium of forces and/or moments along predefined failure surfaces. The approach is widely used due to its simplicity and leads to an adequate and safe design, considered in many codes, by providing a global safety factor. However, it lacks the ability of predicting the kinematic field and the level of stresses, while a main shortcoming is the incapacity of materials to vary their ultimate strength in relation with the stress path. In the second approach, the powerful tool of numerical analysis is applied, providing the ability of modeling the mechanisms developed and allowing for shear strength variation with regard to stress and displacement field. In addition to estimating the safety factor, numerical analysis provides both displacement and stress field, indicating the areas where failure is to initiate and progress. Several numerical analyses under static and seismic loading have been carried out in order to assess the effect of both the multi-stage modeling and the creep action on the response of a reinforced embankment. Quantitative and qualitative comparison of the results arising from the aforementioned approaches is made and the main advantages and drawbacks are discussed.

1 INTRODUCTION

Design of reinforced embankments based on the principles of limit equilibrium is very popular amongst scientific community due to its efficiency in estimating the margins of safety of such a structure in a simple manner, by providing a general factor of safety. On the other hand, the ability of numerical methods to accurately predict the most critical areas and to give information about the kinematic field, on the expense however of high computational demands, is very useful when the design of structures of great extent and importance are concerned. In such cases, design optimization is achieved by strengthening only the weak areas, resulting in safe conditions and cost reduction.

With the aim to evaluate the applicability of the two methods and to compare their results, the same case of a 14.0 m high reinforced embankment is approached by the aforementioned methodologies. The loading conditions included both static and seismic loads, while for the numerical analysis one-stage as well as multi-stage construction were considered. Furthermore, numerical analysis was also carried out to compare the embankment response before and after creeping.

The evaluation of the results of both methodologies leads to interesting conclusions about the applicability limits of limit equilibrium approach on the one hand and the complexity regarding the simulation and interaction of geogrids and fill material involved in numerical analysis on the other.

2 CASE STUDY LAYOUT

The examined case involved a shoulder widening reinforced embankment with a slope of 2:1 (h:b) and a distance between the geogrids equal to 0.5 m. The length and the ultimate resistance of the reinforcement elements were defined using limit equilibrium analysis, after successive iterations which led to an acceptable level of safety. The design section of the reinforced embankment is depicted in Fig. 1, where it can be shown that three soil layers constituted the profile: the rock formation underlying the drain material placed underneath the embankment and the fill material. Except from the geogrids, gabions were placed in the front of the embankment to avoid the development of superficial failure.

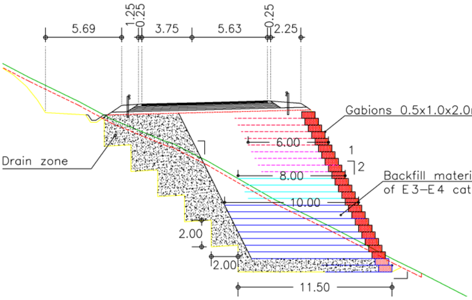

Figure 1. Design section of the reinforced embankment

Table 1. Values for shear strength and deformation parameters of soil layers and geogrids

	Cohesion c (kPa)	Angle of friction φ (deg)	Bulk modulus K (MPa)	Shear modulus G (MPa)	Allowable force Tall (kN)	Axial stiffness (kPa) immediate/creep
Rock formation	50	35	600	450		
Drain material	--	35	90	30		
Fill material	--	30	63	21		
Geogrids 1-12					50	1400 / 500
Geogrids 13-16					34	1272 / 340
Geogrids 17-22					25	1200 / 250

3 LIMIT EQUILIBRIUM METHOD

Limit equilibrium analysis involves three basic checks, after having predefined the failure surface. The first check allows for rotational failure, which in the case of homogenous and isotropic material the failure surface has a circular shape, with its dimensions resulting from the equilibrium between overturning and stabilizing moments. In the case however of reinforced embankments, when the failure surface intersects the geosynthetics, its shape corresponds to a logarithmic spiral (Leshchinsky, 1997). The second check concerns a three part wedge mode of failure and takes into account rigid body equilibrium for the three wedge parts. It should also be considered as complementary to the first check in case of heterogeneous soil formations, such as the rock base formation in comparison with the fill material, as the shape of the failure surface can lead to lower factors of safety. It should also be nodiced that in the case of a non adequate reinforcement disposition, where a surface of failure can be developed in between the geogrids, the analysis may lead to lower values for the factor of safety.

The last check, which practically consists a check against sliding along soil-geosynthetic interface, allows for a translational failure mode (two part wedge mode), where the failure surface consists of two lines, the first of which is along soil-geogrid interface and the second continues inclined to the free surface (Blatz & Bathurst, 2003).

The applied general or partial safety factors play a key role to the design of reinforced embankments. In both cases, the allowable tensile force of the geosynthetics is defined experimentally and after a reduction due to unfavorable effect of creeping, installation damage, time, chemical factors etc.

For the application of limit equilibrium method for the reinforced embankment of Fig. 1 the code ReSSA ver. 2.0 (2006) was used. The design parameters involved are presented in Table 1, while Fig. 2 shows the failure surfaces which correspond to the three aforementioned checks for static conditions and a surface load of 20 kPa. Accordingly, Fig. 3 presents the same surfaces for a seismic action with a horizontal acceleration factor α_h= 0.14 and a surface load of 10 kPa. The factors of safety of the two analyses are given in Table 2.

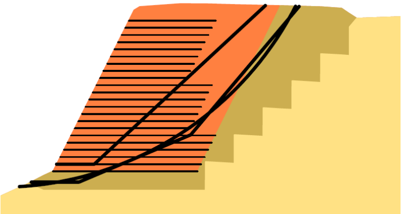

Figure 2. Failure surfaces for static conditions from code ReSSA

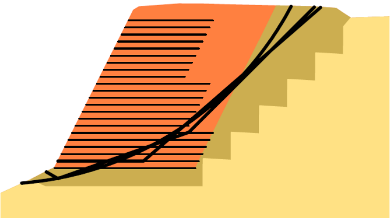

Figure 3. Failure surfaces for seismic action from code ReSSA

Table 2. Factors of safety resulting from the application of the limit equilibrium analysis

	Factors of safety, F	
	Static	Seismic
Rotational failure	1.46	1.15
Three part wedge failure	1.36	1.06
Translational failure (two part wedge)	1.38	1.07

4 NUMERICAL ANALYSIS

4.1 *Simulation Procedure*

For comparison reasons, the same embankment was analyzed using the finite difference code FLAC (2005). The simulation includes the gabion elements for convergence reasons (otherwise the problem does not converge due to steep slope), however their contribution is reduced so as the results of the two approaches to be comparable. The finite difference mesh is depicted in Fig. 4.

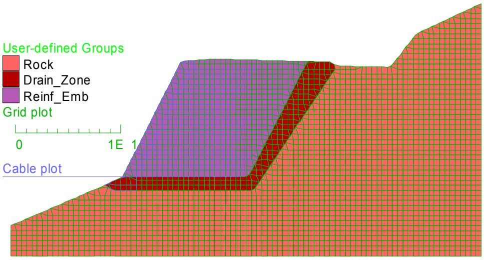

Figure 4. Finite difference mesh

A series of numerical analyses was carried out to investigate the effects of several factors, of which the primary was construction procedure simulation. More specifically, a single construction stage was initially simulated, followed by a multi-stage analysis which simulated the construction procedure with five stages. Although it would be more precise to simulate all the construction phases corresponding to each one reinforcement installation, the computational demands would have been extremely high rendering the numerical analysis less attractive. The analyses were conducted for large strain conditions so as finite element mesh to be readjusted. Axes rotation is implemented to simulate seismic action.

Initial analyses were carried out representing the conditions immediately after the embankment construction. For this case, the allowable tensile force of geogrids is taken equal to the ultimate tensile force, after allowing for several reduction factors, while the axial stiffness is taken directly from laboratory tensile tests. However, axial stiffness is evidently reduced due to creep effects (corresponding to 1.000.000 hrs), and consequently a second series of numerical analyses is conducted, taking into account the aforementioned action. Creep effect is presented in Fig. 5, where behavior before and after creep development is presented (continuous and dash line, respectively). Moreover, the dash-dot line shows the response of a simulation taking into account a reduction in axial stiffness due to creeping from the very first step. In the same figure, a combined simulation response of immediate conditions and behavior after creep appearance is also depicted, represented by a bilinear relationship (bold continuous line). The first branch corresponds to immediate conditions, whereas the second is employed to allow for behavior under creep effect. Obviously, creep effect leads to higher strains, while redistribution of forces on geogrids can take place due to change in kinematic field.

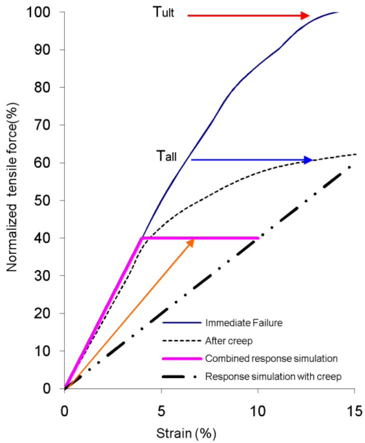

Figure 5. Response and simulation of geogrid under axial loading

4.2 *Numerical Results*

The results of the numerical analysis have shown that displacements are remarkably lower for the upper layers in case of multi-stage analysis (Fig. 7), which can be attributed to the fact that simulation of gradual construction under large strain assumption can reduce the effect of internal settlements. Furthermore, the geogrid forces are approximately 5% lower than in single-stage analysis (Fig, 6), while for the upper layers the above percentage reaches the order of 40%, however the difference is not higher 5 kN.

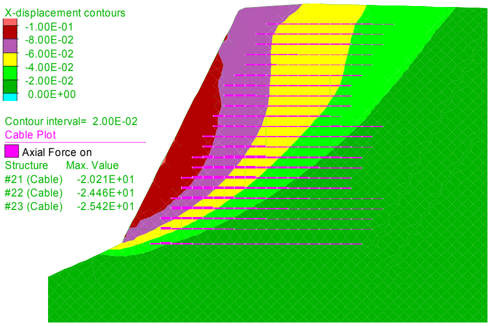

Figure 6. Horizontal displacement field and axial forces of geogrids, single stage analysis

As it can be seen in Fig. 8, in the case where creep effect is taken into account (i.e. the bilinear relationship of Fig. 5 is applied) the displacements are much higher for the entire computational domain, whereas geogrid forces are slightly differentiated to lower or higher values, depending on their location. Analysis under seismic action has led to similar results.

The factor of safety was evaluated for the numerical analysis by continuously reducing shear resistance parameters and was found equal to 1.49 and 1.10 for static and seismic conditions respectively. It is also worth mentioning that numerical results define a failure surface which covers the corresponding areas of failure surfaces specified by ReSSA.

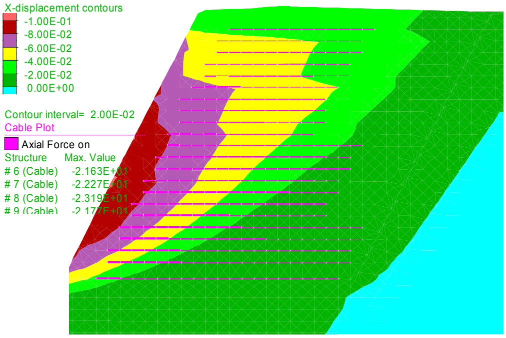

Figure 7. Horizontal displacement field and axial forces of geogrids, multi-stage analysis

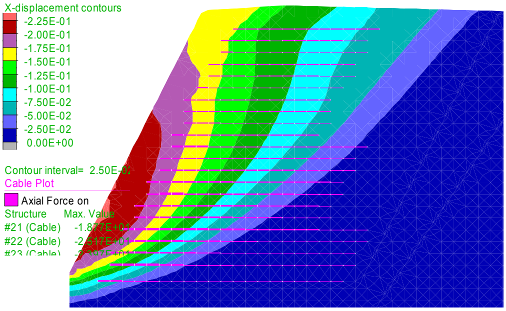

Figure 8. Horizontal displacement field and axial forces of geogrids (analysis under static conditions in conjunction with creep effect)

5 CONCLUSIONS

With the aim to compare the results of limit equilibrium and numerical analysis of a reinforced embankment, the same 14.0 m high embankment was solved using the codes ReSSA and FLAC, for static and seismic conditions. Additionally, the effects of one-stage and multi-stage numerical analysis were investigated and behavior after creep appearance was assessed.

More specifically, although multi-stage analysis is more precise, the simplified assumption of single stage construction leads to satisfactory results for embankments of such dimensions. Moreover, when behavior after creep appearance is examined, the creep effect is significant only for kinematic measures, whereas the forces undertaken by geogrids are slightly differentiated.

The values of factors of safety derived from limit equilibrium method and from numerical analysis allowing for creep effect are comparable. It can therefore be concluded that limit equilibrium analysis, which is widely used given its simplicity, can lead to satisfactory design levels when reinforced embankments of dimensions similar to the case examined are concerned. On the other hand, numerical analysis provides the advantages of predicting both stress and strain distribution which may assist in the conception of complicated mechanisms developed in the case of particular problems where high accuracy is required. In such cases, critical areas showing local sensitivity can be located and reinforced appropriately, resulting in response improvement as well as total cost reduction. However, the high computational demands for such an analysis remain the main disadvantage rendering the applicability of the method value engineering only in special cases.

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