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An analytical study to the vertically reinforcing soil slopes

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ABSTRACT: An approach for stability analysis of vertically geosynthetic reinforced earth structures over firm foundations is presented. The approach involves both internal and external stability analyses. The external stability analysis is based on limit equilibrium through the extension of the bilinear wedge method and it allows a slip plane to propagate between two trenches restrained by sheets of geosynthetic. The internal stability is considered to investigate the pullout capacity of vertical elements through the stable zone of soil. External loadings such as crest surcharge, ground water level and pseudo-static forces of earthquake are included in the equilibrium formulation. In spite of well-established coincidence with experimental data obtained from centrifuge tests this study offers a framework for stability analysis of these reinforced soil structures.

1 INTRODUCTION

Use of horizontal layers of reinforcement in the construction of soil embankments is now a well-developed technique. This method is well suited to the layered nature of new fill construction but cannot be readily employed retrospectively. Barker & Wood (1989) have proposed an alternative method of increasing the stability of existing slopes. This slope reinforcing technique has been devised whereby arrays of parallel vertical sheets of polymer or metal grids, or geotextiles, are installed in narrow trenches, typically 0.2-0.4 m wide, excavated at regular intervals, typically 2-4 m, along a slope. This transforms the slope into an in-situ reinforced soil block, analogues to a soil nailed slope but with an array of high strength sheets rather than bars.

By using this technique, it will be possible to enhance the stability of natural slopes and existing cuttings and embankments without removing large wedges of soils to install arrays of geotextiles and geogrids. This method of reinforcement is chiefly applicable to the prevention, or remediation of shallow slope failures, the occurrence of which is relatively widespread in the over consolidated clays encountered in slopes.

The idea of vertically reinforcing of existing slopes is relatively a new technique among soil reinforcing methods. This technique was first proposed by Barker & Wood (1989), Barker (1991). They used a simple limit equilibrium analysis in a fully drained, dry, cohesionless soil to show that a significant improvement in stability could be achieved using vertical reinforcement. After this various attempts to understand the operation of such structures have been conducted through the centrifugal tests all of them carried out in UK (Jackson, 1998). In these experimental research the effect of vertical elements of geosynthetic has been proved through the limitation of deformations or settlements and improvement of factor of safety against sliding.

This paper summarizes a theoretical investigation into the potential benefits provided to the stability of soil slopes through the action of vertical reinforcement. Outlined in the following are the external and internal stability analyses of a vertically geosynthetic reinforced soil slope over a firm foundation. External stability addresses situations where a reinforced portion may slide as a monolithic block along two parallel reinforcing trenches. Internal stability deals with the resistance to pullout failure within the reinforced soil zone resulting from the interaction between soil and reinforcement. In the framework of this paper, firm foundation implies that deep-seated failures are unlikely to occur and therefore are not considered in the analyses.

2 STABILITY ANALYSIS

2.1 *General*

Limit equilibrium analysis has been used for decades in the design of earth slopes. Extension of this method to the design of vertically geosynthetic-reinforced slopes, where the reinforcement is tangibly modeled, is desirable. The main drawbacks of limit equilibrium analysis are its inability to deal with displacements and its limited representation of the interaction between dissimilar or incompatible materials comprising the slope. Typically, adequate selection of materials properties and safety factors should ensure acceptable displacements, including safe level of reinforcement deformation. The same as unreinforced slopes, factor of safety, F_s, is used to replace the existing slope with artificial one, in which the shear strength is $\phi_d = \tan^{-1}(\tan\phi / F_s)$ and $c_d = c / F_s$ where ϕ_d and c_d are the design shear strength parameter of the artificial soil. Consequently, F_s applies equally to all shear-resisting components be it soil or reinforcement.

Stability analysis in the field of soil reinforcing techniques is generally based on limit equilibrium in which the state of structure respect to the critical condition will be compared with a proper safety factor. On the other hand, inclusion of geosynthetic reinforcement in limit equilibrium analysis is a straightforward process in which the tensile force in the geosynthetic material is included directly in the limit equilibrium equations to assess its effects on stability. To achieve this, several methods may be considered. These include two–part wedge analyses; circular or non-circular analyses and log-spiral failure analyses. A general form of limit equilibrium approach can be developed based on the two-part wedge mechanism. The two-part wedge analysis assumes a bilinear failure surface. This has been shown to provide a reasonable representation of the potential failure surfaces for slopes (e.g., Jewell, 1982). Current code of practice in the UK (BS 8006:1995) introduces this as one of the most commonly used techniques among the methods of assessing stability. The advantage of such an approach is that a wide range of potential failure surfaces can be approximated and the method is relatively simple to program for computer analysis. Therefore in this paper we try to investigate the stability through a two-part wedge mechanism.

2.2 *External stability*

External stability analysis considers the determination of safety factor for a block of soil against sliding along two parallel trenches laterally restrained with reinforcing elements. Figure 1 shows failure surface in this analysis.

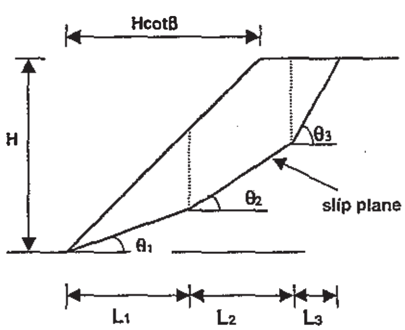

Figure 1. Failure surface in external stability analysis.

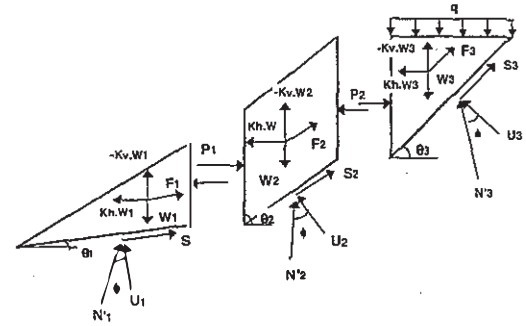

Figure 2. Free body diagram in external stability analysis.

As can be seen, failure mechanism consists of three planar surfaces. Figure 2 shows the free body diagram of forces acting on wedges, where W_i= weight of each wedge; $k_h W_i \,\&\, k_v W_i$=psuedo-static forces due to earthquake in horizontal and vertical direction; F_i=resisting force due to reinforcing elements implies to lateral faces of each wedge S_i=resisting force due to cohesion in base of each wedge; U_i=hydrostatic force due to pore pressure N'_i=induced effective force; P_i=interwedge force and q=crest surcharge.

The assumptions that should be considered are:

1. Soil mass is a homogeneous and isotropic in horizontal plan along the slope.
2. Stresses distribute uniformly along reinforcin elements.
3. Resisting force due to vertical elements impose parallel to the base of failure wedge on the later faces of soil sliding block.
4. The interwedge resultant force P is horizontal.

It is a conservative assumption that implies n shear strain has been developed along the interfac According to Figure 2, unknown quantities interfe ing in the problem are being P_1 ; P_2 ; N'_1; N'_2; N and Fs, where Fs is safety factor against sliding. Fo simplicity the inclination of N'_i is considered equ to ϕ respect to perpendicular on the base failure su face. Now based on the free body diagram shown Figure 2, one can assemble the force equilibrium an angle of $\theta_i - \phi$ respect to horizontal for ea wedge, in which ϕ is internal angle of friction of t soil. Therefore according to active and resisti forces on each wedge , the following expressio may be obtained:

Wedge 1:

$$(S_1 + 2F_1)\cos\phi - W_1(1+k_v)\sin(\theta_1 - \phi) - k_h W_1 \cos(\theta_1 - \phi) - U_1 \sin\phi - P_1 \cos(\theta_1 - \phi) = 0 \quad ($$

Wedge 2:

$$(S_2 + 2F_2)\cos\phi - W_2(1+k_v)\sin(\theta_2 - \phi) - k_h W_2 \times \cos(\theta_2 - \phi) - U_2 \sin\phi + (P_1 - P_2)\cos(\theta_2 - \phi) = 0 \quad (2)$$

Wedge 3:

$$(S_3 + 2F_3)\cos\phi - W_3(1+k_v)\sin(\theta_3 - \phi) - k_h W_3 \cos(\theta_3 - \phi) - U_3 \sin\phi + P_2 \cos(\theta_3 - \phi) - qB(L_1 + L_2 - H\cot\beta)\sin(\theta_3 - \phi) = 0 \quad (3)$$

In above equations, B is distance between trenches and other parameters introduced previously in Figures 1 and 2. In order to determine the safety factor, one can obtain P_1 and P_2 from equations 1 and 3 and then substitute in equation 2, it yields to a value for F_s for a given geometry and failure surface.

It should be mentioned in order to include the effect of pore pressure, a soil parameter called the pore pressure ratio, r_u, is used. The pore pressure ratio is defined as a ratio between the total pore pressure and the total overburden pressure.

2.3 *Resisting force due to reinforcing elements*

A potentially significant problem in limit equilibrium analysis of reinforced soil is the need to know the force in each reinforcing element at the limit-state. In this section, the maximum force transferred to the lateral faces of each wedge from the reinforcement is calculated. It is assumed here that tearing or pullout of the reinforcement is not the critical failure mechanisms.

Many authors (e.g., Jewell, 1980 a, b, McGown et al. 1978), have investigated the performance and behavior of reinforced soil. It has been find that the reinforcement has no initial effect. it is only working after the reinforcement has been strained. As the soil strains it mobilizes strength to resist the shear loads, soil strain causes strain in the reinforcement, which leads to a further increase in strength in the reinforced soil. Reaction of the reinforcement's developed strength acts as a resisting force to the lateral faces of the sliding block of soil.

Based on Figure 3, the maximum force (dF) transmissible to the soil from a small element of reinforcement on each side is a function of the area of that element, the height of soil above it and the factored soil strength parameters. The force transmissible from the area dxdy is:

$$dF = \frac{1}{F_s}\left[k_y h \gamma f_b \tan\phi(1 - r_u) + f_b . c\right] dxdy \quad (4)$$

where F_s=factor of safety against sliding; k_y=coefficient of lateral earth pressure parallel to crest of slope; γ= unit weight of soil; f_b=bond coefficient (a parameter describing the released soil parameters at the soil-geosynthetic interface; typically within the range of 0.6-1.0); ϕ=internal angle of friction of the soil; r_u=pore pressure ratio; c=cohesion of the soil.

The force is summed over the area of the entire wedge. This assumes that the whole of the wedge is located within the zone of reinforcement, an assumption compatible with the shallow failure mechanisms that this technique of reinforcement is designed to counteract.

2.4 *Internal stability*

In reinforced soil structures, the capacity of the reinforcement to develop the required tensile resistance depends also on its pullout resistance; i.e., the length anchored into the stable soil zone (Fig. 4).

The objective of the internal stability analysis is to ensure adequate stability against pulling out for a given length of reinforcement panel, L_r. It is assumed that any reinforcement placed outside the parallelogram section analyzed (to aid drainage, for example) provides no benefit to the pullout resistance.

Based on above paragraph, one can define the factor of safety against pullout for a reinforced slope as:

$$F_p = \frac{F_r}{F_{slip} \cdot \cos\delta} \quad (5)$$

Where F_r=resisting force against pullout; F_{slip} = reaction of maximum transmissible force to the soil; δ = the angle in which the force F_{slip} acts.

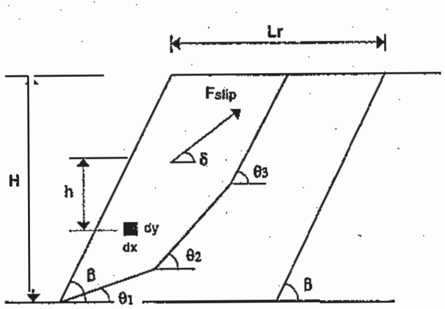

Figure 3. Resisting force due to reinforcing element.

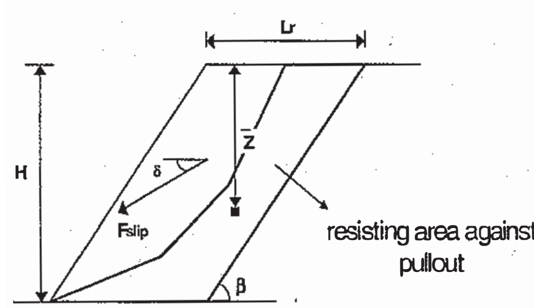

Figure 4. Pullout resistance due to stable soil zone.

According to section 2.3, F_r will be determine from:

$$F_r = \left[k_y \gamma \bar{z} f_b \tan\phi(1-r_u) + f_b . c\right] . S_{stable} \quad (6)$$

Where $\bar{z}$ =the depth of centered of stable area from crest level; S_{stable} = the area of stable zone behind the active zone.

In equation 5, F_{slip}, causes the reinforcement to pull out from the stable zone of the soil. This force is the reaction of resultant force provided by the reinforcement that acts in angle δ respect to horizontal. According to Figure 5, one can determine F_{slip} and δ as:

$$F_{slip} = \sqrt{(\Sigma F_i \cos\theta_i)^2 + (\Sigma F_i \sin\theta_i)^2} \quad (7)$$

$$\alpha = \sin^{-1}\left[\frac{F_1 \sin(\theta_2 - \theta_1) + F_2 \sin(\theta_3 - \theta_2)}{F_{slip}}\right] \quad (8)$$

$$\delta = \theta_3 - \alpha \quad (9)$$

Hence the factor of safety against pullout can be find from equation 5 to 9 for a given length of reinforcement.

3 RESULTS AND CASE HISTORY

3.1 *Typical results*

To analyse the problem a computer program called 'VERSAP' has been developed. This program is written based on concepts and formulation presented in section 2.2 – 2.4 (Davari, 2001). Typical results of VERSAP which use the parameters presented in Table 1 are shown in Figure 6. Firstly, stability analysis is carried out in unreinforced condition. Then analysis is developed for vertically reinforced slope.

According to Figure 6, it appears that vertical geosynthetic reinforcement causes considerable increase in safety factor against sliding. Therefore the

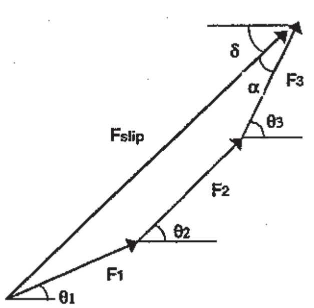

Figure 5. Vectorial representation of forces to determine F_{slip}.

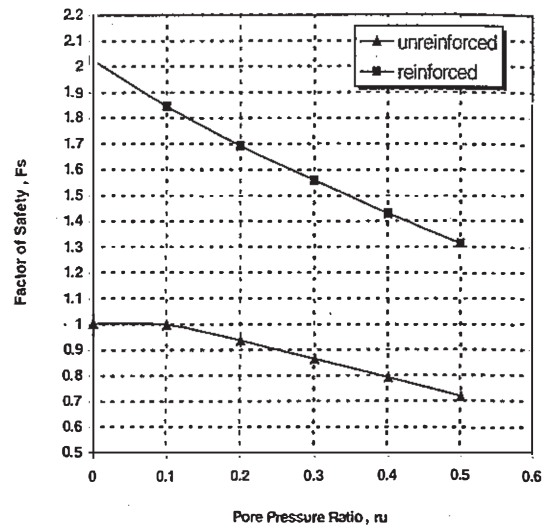

Figure 6. Effect of vertically reinforcing on safety factor.

Table 1. Parameters assumed in analysis of a reinforced slope.

c	φ	γ	β	H	B	ky	q	k_h	r_u	Lr
(kPa)	(°)	(kN/m³)	(°)	(m)	(m)		(kPa)			(m)
10	30	18	45	4	3	0.5	30	0.25	0-0.5	3

proposed analytical and theoretical basis in previous sections has presented a well-effective technique to enhance the stability of soil slopes. Such a result has been arisen from resisting force due to reinforcement included in force equilibrium equations.

3.2 *Case history*

Jackson (1998) reports the results of centrifuge tests of well-instrumented small-scale model slopes. In order to determine the benefits provided by vertical reinforcement in terms of factor of safety, a definition of slope failure was required. He prepared some slope models with vertical arrays of geosynthetic, tested them by centrifuge testing equipment were brought to failure through the increase of centrifugal acceleration (Ng). A factor of safety was established for the slip surfaces in terms of the model scale using simplified form of Bishop's analysis.

The parameters considered in model tests and equivalent prototypes, are summarized in Table 2. The results of VERSAP with safety factors obtained from tests are being shown in Table 3.

The results obtained from theoretical analyses are relatively close to values from experimental investigations and difference between safety factors is nearly insignificant with the view of geotechnical engineering. The good agreement exhibited in Table 3 confirms the accuracy and validity of analytical approach presented in this paper.

Table 2. Parameters considered for testing samples and equivalent prototype models.

Model no.	c	γ	β	H	B	ky	Lr	N
	(kPa)	(kN/m^3)	(°)	(m)	(m)		(m)	
2	41	19.8	60	0.2	0.174	0.575	0.31	64.6
prototype	41	19.8	60	12.9	11.2	0.575	20	-
3	33	19.8	60	0.2	0.105	0.575	0.31	47.1
prototype	33	19.8	60	9.4	5.0	0.575	14.6	-
7	14	18.8	60	0.2	0.075	0.4	0.31	31.5
prototype	14	18.8	60	6.3	2.4	0.4	9.8	-
9	16	18.8	60	0.2	0.105	0.4	0.31	26.0
prototype	16	18.8	60	5.2	2.7	0.4	8.0	-

Table 3. Comparison between test results and computer program VERSAP.

Model no.	Fs from test	Fs from VERSAP	Difference (%)
2	1.41	1.33	-5.7
3	1.56	1.8	15.4
7	1.41	1.42	0.7
9	1.43	1.6	11.9

4 CONCLUSION

A procedure for the design of slopes reinforced with geosynthetic materials has been presented. The analyses involved in the presented design process are based on limit equilibrium. These analyses ensure the reinforced mass is externally and internally stable. The presented design procedure and the results obtained herein have proved that the technique offers a well-effective solution to improve the stability of natural slopes and existing cuttings and embankments. In addition to this, the results from theoretical approach have a considerable coincidence with data obtained by previous experimental investigation. This agreement proves accurately the theoretical basis proposed in this paper.

The proposed design procedure can be easily carried out using a computer program (e.g., Davari, 2001). The mechanism and analysis used can be replaced with other rigorous stability methods. Hence, this paper provides a conceptual framework for design of vertically geosynthetic reinforced slopes.

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