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CHRISTIE I.F. and EL HADI K.M.
University of Edinburgh, Scotland

Some aspects of the design of earth dams reinforced with fabric

Quelques aspects de la conception des barrages en terre, armés de textile

On decrit une methode pour dessiner des barrages de sol, arme de textile. La contribution de l'armature est comprise dans le calcul du facteur de securite contre glisser sur une ligne de glissement circulaire. On discute aussi la possibilite d'employer de l'armature pour empecher le fendillement des terrassements en remblai de sol argileux. Compris aussi sont quelques resultats experimentaux des resistances au cisaillement entre des sol coherants et des textiles.

INTRODUCTION

In recent years considerable use has been made of the reinforced earth technique developed by Vidal and others (Schlosser and Vidal 1969). In most of the applications conventional retaining walls have been replaced by reinforced earth walls comprising metallic strip reinforcement and metal or concrete facing units (Price 1975). A similar type of construction has been used for a small overflow dam in France (Corda 1973).

In this paper consideration is given to the possible use of continuous sheet fabric reinforcement in embankments with non-vertical sides.

STABILITY OF A REINFORCED EARTH DAM

In the design of a reinforced earth dam an adequate factor of safety must be provided against the following possible modes of failure:

(1) Sliding on the base, treating the entire reinforced mass as a rigid body.

(2) Failure of the reinforcement in tension.

(3) Shear failure through the reinforced embankment.

(4) Failure of the reinforcement through inadequate bond length.

Sliding on the base

Assuming that the embankment is composed of free-draining granular material, an impermeable membrane must be provided. To minimise the volume of fill this should be constructed at the upstream face.

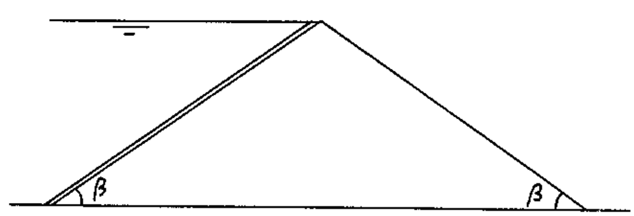

Fig. 1 Triangular dam with impermeable membrane at upstream face.

For the simple triangular profile shown in Fig. 1, it can be shown that the factor of safety against sliding on the base is given by

$$F_1 = \frac{\mu}{\tan\beta}(1 + 2\gamma/\gamma_w) \qquad (1)$$

where μ = coefficient of friction between fill and underlying soil or rock ($\mu \not> \tan\phi$ where ϕ = angle of internal friction of the fill material)

γ = unit weight of fill

γ_w = unit weight of water

If μ is approximately equal to $\tan\phi$ and $\beta = \phi$, $F_1 \doteqdot 5$. Alternatively, $F_1 = 2$ when $\beta \doteqdot 60$. Practical designs, incorporating freeboard above top water level and a horizontal profile at the top, will have a higher value of F_1. Hence this type of failure is unlikely to govern the design unless the underlying foundation material has very low shear strength.

Spacing of reinforcement.

In choosing a suitable spacing for the reinforcement it may be assumed tentatively that this should be capable of withstanding the horizontal component of active earth pressure in an infinite slope.

A similar assumption has been used in the design of reinforced earth retaining walls with horizontal backfill (Lee, Adams and Vagneron 1973). Higher pressures, corresponding to 'at rest' earth pressures, have been deduced from full-scale measurements of the tensile force in the reinforcement of such walls (Chang 1974). However, in the present context, this may be countered by the overestimate involved in the assumption of an infinite slope.

In the case of a cohesionless fill, the coefficient of active earth pressure, K_A, is given by Rankine's theory as

$$K_A = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

At depth z below the surface of the slope the horizontal component of active pressure is

$$K_A \gamma z \cos^2\beta$$

We shall assume the reinforcement to form continuous horizontal sheets at intervals of ΔH vertically. Each sheet has a tensile strength T_1 per unit width. The factor of safety against failure of the reinforcement in tension is then

$$F_2 = \frac{T_1}{K_A \gamma z \cos^2\beta . \Delta H} \qquad ......(2)$$

Shear failure of reinforced embankment

The possibility of shear failure along a continuous surface through the embankment may be analysed by the usual slip circle method of slices, modified to take account of the tension provided by the reinforcement.

The shear strength (τ) of the fill material may be defined generally in terms of the Mohr-Coulomb failure criterion, i.e.

$$\tau = c + \sigma_n \tan\phi$$

where c = cohesion per unit area

σ_n = normal stress across the failure plane

Alternatively, in terms of effective stresses,

$$\tau = c' + (\sigma_n - u)\tan\phi'$$

where u = pore water pressure.

The reinforced mass above an assumed circular slip surface of radius R may be divided into a number of vertical slices. At the level of the base of a slice, the reinforcement is capable of providing a horizontal force T ($= T_1/\Delta H$) per unit height. Neglecting forces on its vertical sides, an approximation which leads to an underestimate of the factor of safety in unreinforced slopes (Bishop 1955), and applying the same factor of safety to the tensile strength of the reinforcement as to the shear strength of the soil, the forces on one slice are as follows (see Fig. 2):-

W = total weight of slice

N = total normal force across base

$\tau l/F$ = shear force on base of length l

Td/F = force from reinforcement across the base

where F = factor of safety against failure in this mode.

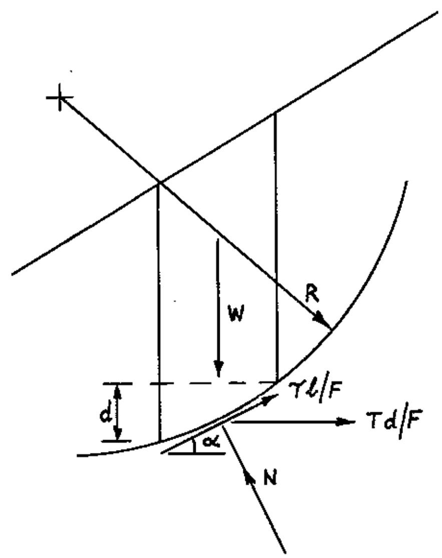

Fig. 2 Forces acting on typical slice above assumed slip circle.

Taking moments about the centre of the circle, and summing over all slices,

$$\Sigma W R \sin\alpha = \Sigma(T\ell R/F + Td \cos\alpha R/F)$$

where α = slope of base of slice

Also $T\ell = c\ell + (W\cos\alpha + Td\sin\alpha)\tan\phi$

Hence,

$$F = \frac{\Sigma\{c\ell+(W\cos\alpha + Td\sin\alpha)\tan\phi + Td\cos\alpha\}}{\Sigma W\sin\alpha} \quad \ldots(3)$$

The corresponding expression in terms of effective stresses is

$$F = \frac{\Sigma\{c'\ell + (W\cos\alpha - u\ell + Td\sin\alpha)\tan\phi' + Td\cos\alpha\}}{\Sigma W\sin\alpha} \quad \ldots(4)$$

In these equations α , and hence sin α , may be negative for some slices.

Bond failure of reinforcement.

Failure of the reinforcement through inadequate bond length will not arise if the sheets extend across the full width of the cross-section . Otherwise the bond length should be checked at various points around the critical slip circle.

Using the active earth pressure assumption, the force to be resisted by a reinforcing sheet at depth z below the surface of a cohesionless slope is

$$K_A \gamma z \cos^2\beta \, . \, \Delta H$$

If the sheet extends inwards a distance L from the point considered, the pull-out resistance is likely to be somewhat greater than $2L\gamma z \tan\psi$

where ψ = angle of friction between fill and reinforcement

Hence, the factor of safety against lack of adherence is

$$F_4 = \frac{2L \tan\psi}{K_A\cos^2\beta \, . \, \Delta H} \quad \ldots\ldots\ldots\ldots(5)$$

Illustrative example

As an example of the application of the above criteria the embankment shown in Fig. 3 will be considered.

The soil is assumed to have the following properties:-

γ = 19.6 kN/m^3

c = 0

ϕ = 35^o

The tensile strength of the reinforcing sheets is 50 kN/m width.

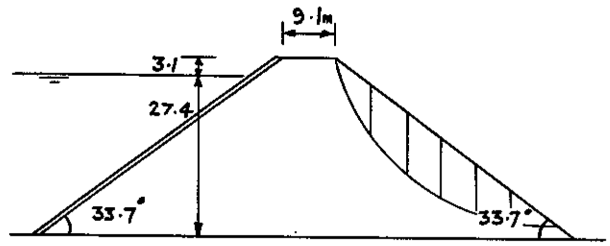

Fig. 3 Cross-section of dam analysed in design example.

Assuming an angle of friction between fill and foundation material of 30^o, the factor of safety against sliding on the base = 6.0.

The provision of adequate reinforcement and the calculation of the factor of safety against failure on a circular slip surface requires to be checked for numerous possible circles only one of which, shown in Fig. 3, will be considered here.

The area above the assumed slip circle is divided into a number of vertical slices and the horizontal component of active earth pressure is calculated at the midpoint of the base of each slice. From equation (2) the vertical spacing of the reinforcement can then be calculated at various elevations to satisfy a specified value of F_2. A value of $F_2 \doteqdot 1$ has been used for the calculations shown in Table 1.

TABLE 1 Calculation of reinforcement required to satisfy equation (2)

Slice	Height of slice (m)	Horiz. press. (kN/m^2)	Reinft. ΔH(m)	No.
1	6.4	60.8	0.8	16
2	10.4	98.8	0.5	13
3	11.6	110.2	0.45	9
4	11.6	110.2	0.45	6
5	9.1	86.5	0.55	3
6	4.0	38.0	1.3	1

The factor of safety against sliding on the slip circle shown, calculated from equation (3), is 2.06. This may be compared with a value of 1.23 neglecting the reinforcement, or with the critical value for this slope if unreinforced of 1.05 (= tan ϕ/tan β).

Equation (5) has been used to check the bond length required to the left of points on this slip circle.
Assuming F_4 = 2 and $\psi = \phi$, the required

length = $0.7\,\Delta H$, giving values of less than 1 m.

COHESIVE SOILS

In some areas it may be more economical to construct the embankment with cohesive soil, or with a cohesive clay core forming the impermeable barrier together with frictional soil or rockfill shoulders.

As indicated above, horizontal reinforcement should increase the general stability of an embankment. Additionally, suitably placed fabric might inhibit crest cracking and hydraulic fracturing and piping in cohesive soils.

Crest cracking

Tension cracks may occur at the crest of a dam due to tensile strains associated with differential settlement along the longitudinal axis of the dam (Leonards and Narain 1963). This situation is shown diagrammatically in Fig. 4.

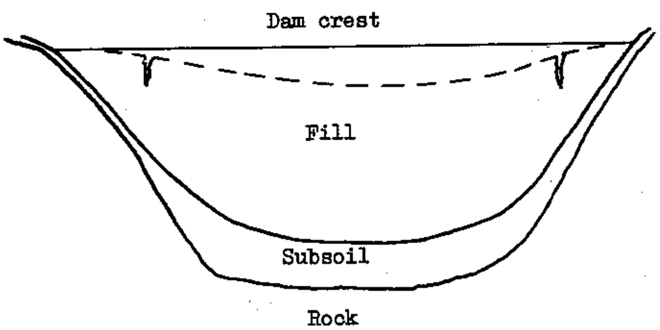

Fig. 4 Tension cracks formed by differential settlement along longitudinal axis of dam.

Some indication of the feasibility of using reinforcement to resist cracking can be obtained by comparing the tensile strength of reinforcement per unit area served by it ($T = T_1 / \Delta H$) with the tensile stresses which have been estimated or observed in existing dams.

For example, at Rector Creek Dam (Leonards and Narain 1963), tensile strains of 2.4×10^{-3} were observed and the modulus of elasticity of the soil, measured in beam tests, was approximately 19 MN/m^2. Horizontal tensile stresses near the crest could be expected to be about 45 kN/m^2. Neglecting the tensile strength of the soil and allowing for a factor of safety of 1.5, the required strength of fabric would be 67.5 kN/m^2 of area served by it, or 34 kN/m if spaced at intervals of 0.5 m vertically.

As this reinforcement would be near the top surface, very little overburden pressure would exist to develop bond strength between the soil and the fabric. Consequently the need for some form of anchorage would have to be investigated.

Hydraulic fracturing and piping.

In dams with a narrow central clay core, arching of the compressible core may occur between the more rigid upstream and downstream shoulders with the result that the values of total vertical stress in the core may be less than the conventional overburden pressures. Assuming the soil is unable to withstand tension, horizontal cracks will develop at elevations where the total vertical stress is less than the pore water pressure created at the upstream side of the core when water is impounded in the reservoir (Kjaernsli and Torblaa 1968).

Theoretically it would be possible to include fabric reinforcement in the core to withstand such negative effective stresses but the practical difficulty of placing sheets of fabric vertically in the core would have to be overcome.

Possibly a simpler use of fabric in this context would be to provide a layer on each side of the core to prevent piping, or erosion of fine soil particles, into the adjacent coarse soil, or rockfill, shoulders. In this case the fabric would replace conventional soil filter zones. Similarly, fabric might be used in place of soil filter zones adjacent to horizontal drainage layers within the embankment.

SHEAR BOX TESTS

With special reference to cohesive soils, tests are in progress to obtain information on the soil-fabric adhesion and skin friction and the filtration properties of various fabrics. Some results of the tests to determine adhesion and skin friction are given below.

Shear box tests have been carried out with various compacted cohesive soils and fabrics. In each case soil at a particular moisture content was compacted in a mould to give the same density as obtained at that moisture content in the standard Proctor Compaction test. A group of such samples was then subjected to conventional quick shear tests in a 6cm x 6cm shear box.

In another group of tests the lower half of the shear box held a wooden block on top of which was glued a sample of the fabric to be tested. The upper half contained a soil sample prepared as described above.

The following fabrics were tested:-

Fabric A: Woven tape polypropylene, 60

tapes (1000 denier)/10cm warp and weft.

Fabric B: Spunbonded polypropylene - nylon, 140g/m^2.

Fabric C: Wire reinforced jute scrim, 0.375mm mild steel galvanised wire plied with jute, 50 x 35 threads/10cm.

Three cohesive soils have been used in the tests to date, one sandy clay and two highly plastic clays.

The results for the sandy clay (LL = 28%, PL = 17%), tested at a moisture content of 16.25% (1.55% above Proctor optimum), are shown in Fig. 5. From this it can be seen that the adhesion (i.e. shearing resistance at zero normal stress) and skin friction developed by Fabric C is equal to the cohesion and internal friction of the soil. The shearing resistances developed with fabrics A and B are only slightly less than this.

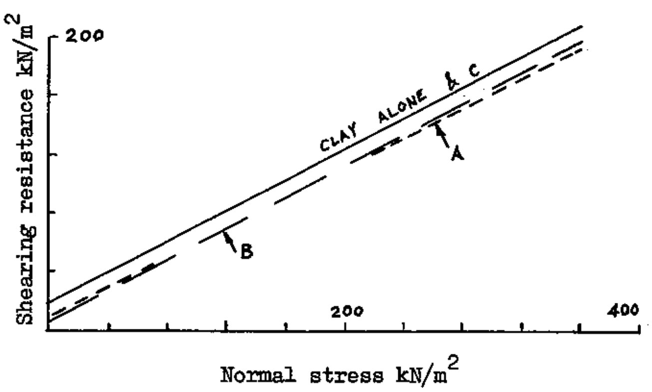

Fig. 5 Results of shear box tests with sandy clay in contact with Fabrics A, B and C.

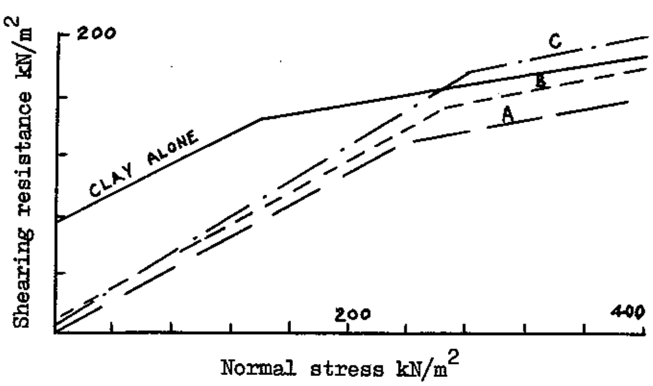

Fig. 6 Results of shear box tests with London Clay in contact with Fabrics A, B and C.

Tests were carried out at various moisture contents with London clay (LL = 69%, PL = 29%). The results obtained at a moisture content of 26.7% (just above optimum) are illustrated in Fig. 6.

The shape of these graphs is similar to those reported for clay soil in contact with other construction materials (Potyondi 1961). Again Fabric C exhibits slightly greater shearing resistance than the other fabrics tested. With all three fabrics the adhesion is small in relation to the cohesion of the clay, but at high values of normal stress the shearing resistance of the soil-fabric system is similar to that of the soil alone.

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