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Fabrics below sand embankments over weak soils, their technical specifications and their application in a test area

Textiles sous remblais, spécifications techniques et application expérimentale

Résumé

Textiles sous remblais, specifications techniques et leur application en forme expérimentale.

A l'aide d'um modèle simple de calcul des spécifications techniques pour l'emploi des textiles sous un remplai ont été developpées. Le modèle tient compte des contraintes dans le plan du membrane et le poids statique du remblai. Les spécifications sont basées aux fonctions suivantes du membrane.

La force portante du membrane doit être suffisante à porter seul la première couche du remblai jusqu'au moment que, par tassement, le remblai touche le sous-sol plus résistant au dessous de la vase. On rencontre ce cas aux Pays Bas durant le remplissage des fossés nombreux.

Le courbe effort-élongation du membrane doit être telleque le membrane contribue localement au force portante du sous-sol peu résistant.

Membranes textiles qui satisfont aux spécifications décrités ont été employés en forme expérimentale sous un remblai pour une autoroute. Les premiers résultats montrent que la présence d'un membrane augmente un peu au coëfficient de sécurité du remblai.

Fabrics below sand embankments over weak soils Technical specifications and their application in a test area.

1. Introduction

Construction of embankments of a rapid rate, as takes place in hydraulic filling, often causes shearing and displacement of weak soils with a poor bearing capacity. Moreover, weak spots in the subsoil are betrayed by strong differences of settlement. Such unwelcome phenomina do not only cost additional sand, but also effect a quality decrease of the embankment.

Shearing planes often develop from former ditches and weak spots in the subsoil. Great differences of settlement appear principally in and near former ditches. It is assumed that a load-spreading layer may contribute to the stability of the embankment and may prevent settlements from varying too widely.

Previously to the construction of a number of test areas with fabric sheets below sand embankments quality specifications for the fabrics to use have been developed. For this purpose the possible functions of fabrics below a sand embankment mentioned above have been quantified in simple calculation models that will be discussed below.

2. Estimation of force and deformation

The bearing capacity of a fabric sheet in comparison with that of the subsoil itself is negligible. Hence it is clear that a sheet placed in a cutting about 60 m in width cannot support the embankment that has to be filled upon it. For temporary bridging a rather wide ditch, however, the bearing capacity of a fabric sheet may be sufficient. The subsoil below a ditch is not only often very poor, but the embankment over de ditch is, moreover, thickest and the load accordingly greatest. If synthetic fabric sheets are used as a load-spreading expedient it is obvious to assign a part to them in bearing the additional load at the spot of the ditch. Especially during the filling operation the sheet has to bear the extra load on the bottom of the ditch.

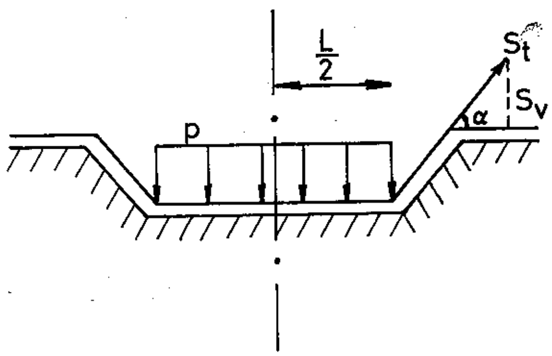

figure 1

Figure 1 shows that the tensile strength S_t has to fulfil the equation $S_t \geqslant \frac{pL}{2\sin\alpha}$, if the load on the slopes of the ditch and the friction between the sheet and the subsoil are neglected. The sand fill in a common ditch (depth 1m, bottom width 1.50 m) if submerged exerts a pressure p = 0.01 MN/m^2. The sheet has therefore to develop a tensile strength of 10.6 kN/m' if the slope is 1 : 1 and 16.6 kN/m' if the slope is 1 : 2. To avoid making requirements too severe one could fix the mean tensile strength at 13 kN/m'. The decrease of the tensile stress by creep of the sheet has not to be taken into account. The load of the ditch fill has to be borne only during a short period, i.e. until the strain of the sheet and displacement of the mud on the bottom of the ditch results in a direct support of the fill by layers of greater bearing capacity (peat). In the foregoing a configuration in which the sheet is laid through a cutting has been discussed. In general an unrolled sheet will cover a week spot in the subsoil nearly horizontally. In that case the membrane function of the sheet has to supply the required bearing capacity.

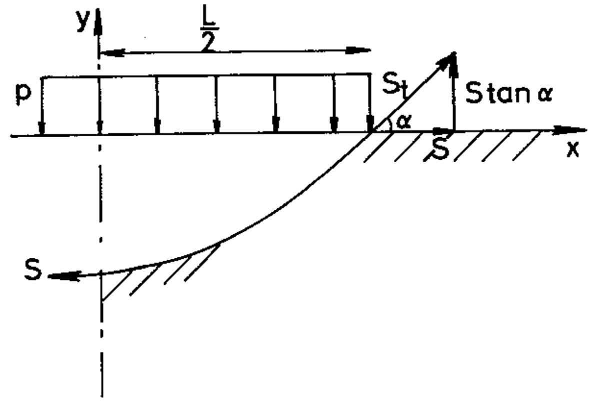

figure 2

In figure 2 a sheet deflecting under a uniform load p is shown. On closer examination of the calculation model for safety the bearing capacity of the weak spot is neglected, at least for relatively small loads yet to be defined.

If the friction between the sheet and the subsoil and the weight of the sheet itself are neglected, the horizontal component S of the force acting in the sheet is constant. The vertical component S tan α makes in a point x = x equilibrium with the vertical load px:

$$S\tan\alpha = S\frac{dy}{dx} = px. \qquad (1)$$

The force S_t in the direction of the tangent to the sheet equals:

$$S_t = S\sqrt{1+\left(\frac{dy}{dx}\right)^2} = S\sqrt{1+\left(\frac{px}{S}\right)^2}. \qquad (2)$$

By integration of (1) we find for the deflection y of the sheet

$$y = \frac{1}{2}\frac{p}{S}x^2 - \frac{1}{8}\frac{p}{S}L^2. \qquad (3)$$

On the hypothesis of a linear elastic behaviour of the sheet the resulting strain can be calculated as:

$$S_t = E'\frac{du_t}{dx}, \qquad (4)$$

in which E' = stiffness modulus in kN/m', u_t = displacement of a point x of the sheet in the plane of the sheet as a result of the strain.

Using (2) from (4) can be derived:

$$du_t = \frac{S}{E'}\sqrt{1+\left(\frac{px}{S}\right)^2}\,dx \approx \frac{S}{E'}\left(1+\frac{p^2x^2}{2S^2}\right)dx. \qquad (5)$$

The approximation causes only little errors if the value of $\frac{dy}{dx}$ is low. Integration of (5) gives:

$$u_t = \frac{Sx}{E'} + \frac{p^2x^3}{6E'S} \qquad (6)$$

$u_t(x = \frac{L}{2})$ is the extension of the strained sheet, since $u_t(x = 0)$ equals zero. The extension of the sheet can also be calculated directly from the length of are of the deflected sheet

$$\frac{L}{2} + \Delta L = \int_0^{\frac{L}{2}} \sqrt{1 + (\frac{dy}{dx})^2}\, dx \approx \frac{L}{2} + \frac{p^2L^3}{48S^2}. \qquad (7)$$

The extension ΔL must be equal to $u_t(x = \frac{L}{2})$ from (6) so that the following equation applies:

$$S^3 + \frac{p^2L^2S}{24} - \frac{p^2L^2E'}{24} = 0. \qquad (8)$$

Equation (8) gives a relation between the still unknown horizontal force S in the sheet and the parameters p (load), L (width) and E' (stiffness modulus). There has to be chosen such a combination of S and E', that by strong differences of settlement the sheet contributes to the bearing power without the maximum value of S_t (for $x = \frac{L}{2}$) exceeding the tensile strength of the fabric.

To contribute to the load spreading, the sheet at least has to bear the additional heigth to eliminate the differences in settlement. For, filling up and profiling an embankment more sand will be putted down at the spots with the largest settlement. The fabric sheet has to be able to eliminate these differences in load. Suppose a rather large difference in settlement of 0.30 m over 1.50 m. If $y_o = 0.30$ m, the maximum load is $p = 6$ kN/m^2 (the unit weight of wet sand is about 20 kN/m^3). For L = 1.50 m and a uniformly distributed load equated to the maximum load for safety from (3) can be calculated S = 5.6 kN/m'. According to (2) the maximum force $S_t(x = \frac{L}{2})$ amounts to 7.3 kN/m'. From (8) the stiffness modulus E' is computed at 57 kN/m'. The maximum strain $du_t/dx(x = L/2)$ can be determined from (5) and amounts 0.13. So, the fabric sheet is able to bear the additional loads as a result of settlements by a maximum difference in settlement of 0.30 m over 1.50 m if at a strain of 13% the stress in the sheet may amount at least 7.3 kN/m'.

Repeating the calculation for settlements smaller than 0.30 m and L = 1.50 m leads to the relation between $S_t(x = L/2)$ and $du_t(x = L/2)$ shown in figure 3 under (a). However, it is evident that also L varies with y_o. Generally L will increase by y_o. Assuming y_o/L^2 is constant from (3) can be derived that p/S is also constant. If L varies in this way the relation between $S_t(x = L/2)$ and $du_t(x = L/2)$ becomes as given in figure 3 under (b).

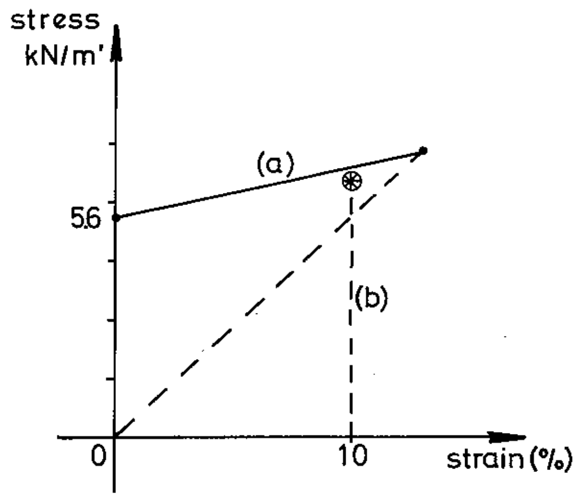

figure 3

Because of the roughness of the calculation-model and the relative arbitrereiness of the numerical values chosen it will be the best to determine an average value from figure 3. To use this value as quality criterion it has to be lie preferably not too far from the point of intersection of both curves because the activity of the sheet especially at larger settlements has to be secured. Considering a strength of 6.5 kN/m' by a strain of 10%. It is desirable that the sheet is able to stand this strength for a large time. The decrease of the stress by creep of the sheet has to be taken into account certainly for the period of settlement of the consolidating subsoil. By way of physical-mechanical measurements to fabrics it is known that to bear a stress permanentely, this stress has to be approximately two times as strong as the stress by a strain of 10%. One thing and another results in a tensile strenght minimal required of 13 kN/m'. This value agrees with the requirement of the tensile strength mentioned before with respect to the bearing of the load of ditch fills.

Because of the capacity to withstand tensile forces the fabric may act a part in the increase of the stability of an embankment. For more details concerning the replacement of the sheet in the stability calculation is referred to the paper "Mechanical behaviour of membranes in road foundations" by J.G. Bakker. According to this calculation model the improvement of the stability factor may be 5 or 10% for sheets with a tensile strength of about 15 kN/m'.

By virtue of the simple calculation models mentioned above, the next stress-strain demands are made for the use of fabrics below sand embankments.

a. The stress has to be at least 6.5 kN/m' by a strain of 10%.
b. The tensile strength has to be at least 13 kN/m' by a strain of 15% at least.

3. Test stretches

To try out the possible functions of fabrics below sand embankments seven test stretches of 120 x 50 m^2 have been constructed in the embankment for the widening of a State Highway. In five of these test stretches fabrics of the quality requirements mentioned were laid between the subsoil and the sand fill. The other two stretches act as a blank for the measuring results. On behalf of finding expression to the contribution of the membrane to load spreading, test stretches provided with ditches are chosen.

To investigate the contribution of fabrics to the stability, the testplan is based on the creation of differences between the stretches with and without membrane, clearly perceptible in the field. That is possible by loading the subsoil till the ultimate bearing capacity. The height of the first hydraulic filling has to be chosen so that there is still just equilibrium (= just no equilibrium). This critical height or critical load can be determined from a stability calculation by persueing a stability factor of 1.0. Practising this criterion of equilibrium may cause failures of slopes of the test stretches without membrane. If the contribution attributed to the fabric is possitive indeed, the reinforced stretches are not permitted to show any unstability by the critical height.

With respect to the distinguishing of possible differences in settlement between the stretches the total height of the embankment of 2.50 m will be considered, since this phenomenon may make perceptible at first after constructing the foundation of the road.

The stability calculations were done for the cross-section drawn in figure 4. In this computation the membrane is replaced by a thin cohesive layer of 0.01 m thickness and $\gamma = 10$ kN/m^3, $\varphi = 0$ and a cohesion based on the tensile stresses acting in the membrane. Since the turf is also able to bear a tensile force, in the calculation it is replaced by a clay layer of 0.10 m with a cohesion based on a tensile force of 0.3 kN/m'. On the bases of experiments and calculations the critical height of the sand layer for the stretches without membrane is determined to 1.25 m. From measurements is found that the height of the first hydraulic filling varies between 1.7 m and 1.1 m with an average thickness of 1.30 m. The path of the phreatic table is based on observations and experience. With respect to the excess pressure in the coherent layers as result of the load of the sand fill, the direct consolidation has been estimated at 50% on the bases of waterstress measurements. As a compromise between calculation and observation slide circles are considered emerging 4.5 m behind the dam, tangent to the top of the pleistocene sand and almost including the entire ditch.

4. Results of stability calculations and test stretches

By means of stability calculations for sand thicknesses of respectively 1.3 m, 1.7 m and 2.5 m, the relation between the stability factors (η) and the necessary tensions in the membranes has been determined.

Considering one and the same slide-circle leads to a linear relation (figure 5). The drawn lines are obtained by calculation and the stippled lines by interpolation. From figure 5 appears, that the tension in the membrane has a relatively

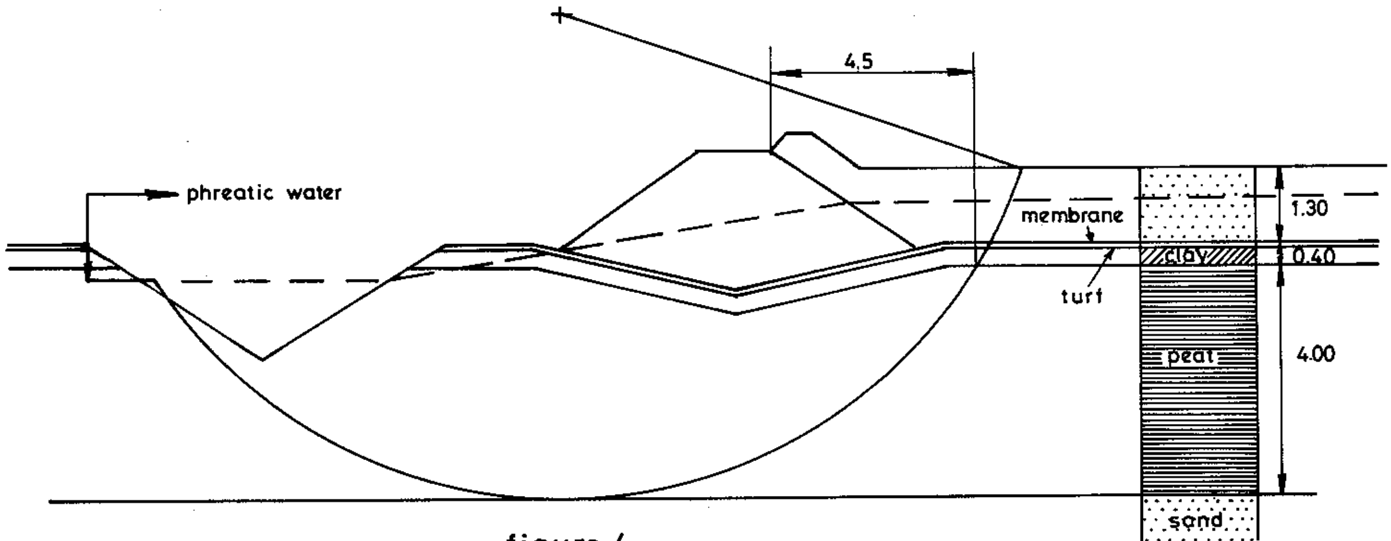

figure 4

small influence on the stability. Loss of stability has been established in both test stretches without membranes by thicknesses of layer of sand greater than about 1.30 m, while the stability in the test stretches with membranes still is maintained at heights of 1.40 to 1.45 m. Therefore it is acceptable to fix the critical thickness of layer of sand at about 1.30 m, or by a thickness of layer of sand of 1.30 m at the test stretches without membranes the real stability factor amounts 1.0. According to figure 5 a calculated stability factor of 0.85 goes with the concerning thickness of layer of sand on a test stretch without membrane. As result of the approximate character of the calculationmethod together with the inaccuracies

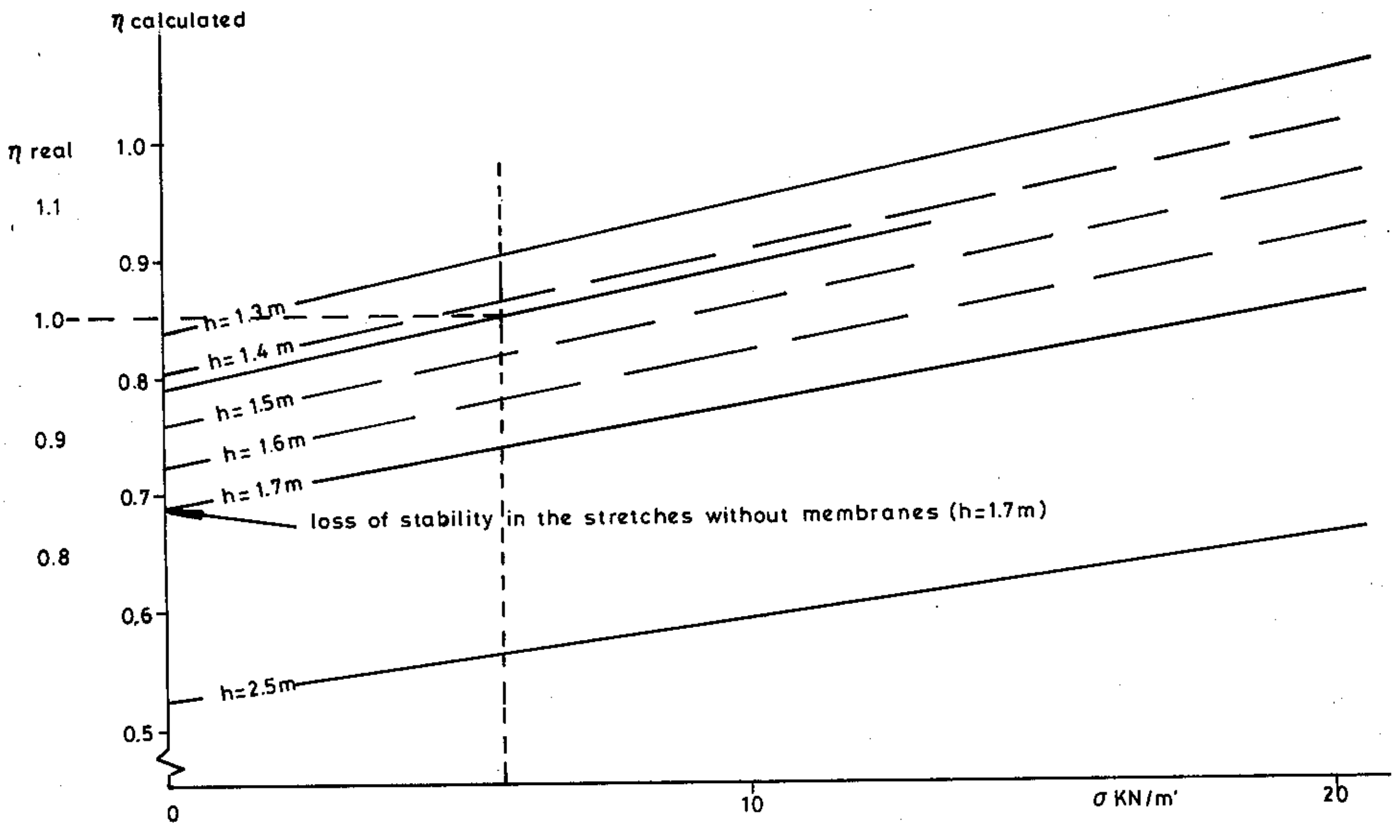

figure 5

in the introducted parameters, η -calculated apparently is to low. The stability of stretches with membranes at a thickness of layer of sand of 1.30 m ($\eta > 1$), while accepting the calculated η-values, is according to figure 5 only possible if the tension in the membranes is greater than 15 kN/m', what is unlikely high. Because there is little known of the tensions in the membrane, the exact difference between the "real" and calculated stability factor can not be determined from the "armed" stretches. It seems more correct to base the difference on the stability criterion in the stretches without membranes. The fact is, that the η-real of 1.0 corresponds with η-calculated of 0.85. This creates, according to figure 5, a situation, that approaches closely to the reality, namely the stretches without membranes with h > 1.3 m are clearly unstable and the stretches with membranes, with h = 1.40 m to 1.45 m, are stable with fairly strong tensions in the membranes. Based on an average η-real value of 1.00 with an average height of 1.43 m there have to be tensions in the membranes from approximately 5.75 kN/m' (figure 5). Dependent on the type of fabric a strain of respectively 1.5% (A), 4% (B) and 6% (C) corresponds with this tension. On the one hand, the strain is caused by the elastic horizontal deformations of the coherent layers under the influence of the load and on the other hand by deformations with loss of stability (modeling slide plane). On account of measured horizontal displacements of the embankment and calculations is determined, that on the position where the slide circle will intersect the original ground surface, the strain in the membrane (= horizontal deformation of coherent layers) can amount to an average of 2.2%.

Hence for material type A (1.5%) the prestress by deformation without loss of stability is sufficient. For type B, respectively C from the σ-ε-relations available is found that an average strain of 2.2% provides a prestress of 3.5 kN/m', respectively 1 kN/m'. The remaining stress of 2.25 kN/m', respectively 4.75 kN/m' has to be produced by an extra strain of 1.8%, respectively 3.8%. We suppose that this extra strain will take place in a very limited zone on both sides of the intersection of the (potential) slide plane and the membrane. It is assumed, that the local stress peak in the membrane is carried by the shear resistance on both sides of the membrane which may decrease lineairly with the distance. The maximum deformation of the contributing zone along the membrane can be computed at 0.005 m (type B), respectively 0.015 m (type C). It is imaginable that on the spot of a potential slide plane deformations of this size may be effected without forming a crack. According to the same reasoning an embankment of 1.70 m will require a stress in the membrane of 24 kN/m' with deformations of 0.05 m at least. It is impossible that such deformations will appear without cracking of the subsoil, if the tensile strength of the membrane has not been exceed yet.

5. Conclusions

With respect to the functions ascribed to fabrics below sand embankments the contribution to the stability has been discussed. To give an idea of the behaviour of the test stretches concerning the difference in settlement the data needed are lacking.

In respect of the contribution to the stability from the measuring results the next conclusion may be drawn.

- The application of a fabric with a stress-strain relation that agrees with the relations of the membranes used, does not improve the stability of an embankment to such an extent, that the thickness of the first hydraulic fill will be essentially bigger than the usual thickness. The membrane may only improve the stability considerably if the fabric is able to develop a much higher strength by substantial smaller deformations.