# Effect of surface reinforcing of soft soils

## Effet du armement superficiel des sols moux

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SYNOPSIS The behaviour of reinforced footing on soft subsoils has been analyzed by the finite element method. The soil has been treated as a hypoelastic-plastic body and the reinforcement as an elastic membrance. Linear elements of the reinforcement are compatibly bound with the triangular mesh of the soil body. Data of some model tests have been used for the verification of the computer program. From the application of the program for a road embankment lying on a soft clayey layer the following conclusions could be drawn:

(1) The geotextile membrance restrains the lateral displacements of the subsoil, favourably modifies the displacement pattern reducing somewhat maximum settlements and increases the bearing capacity.

(2) In a poorly permeable undrained soils the reinforcing effect is particularly pronounced in the initial consolidation phase of prevailing distortional displacements.

(3) The finite element method is a useful tool for predicting the reinforcing effect.

## INTRODUCTION

In order to increase the bearing capacity of soft soils under road embankments the transverse reinforcement of the footing has been applied for hundreds of years. In the past, trunks, branches or fascines were mainly used for this purpose. In recent years metal reinforcing grids have also been used to some extent. However, geotextiles seem to have replaced most of the ancient materials, at least in the industrialized countries (Eggestad, 1983).

In the past such strengthening constructions were governed only by experience. Modern trends are to put criteria for their need and suitability on the base of computation procedures that take into account soil and reinforcement properties as well as the conditions for their cooperation.

In this paper we shall endeavour to contribute to the elaboration of such criteria through a finite element analysis. The soil will be treated as a hypoelastic-plastic body with the Mohr-Coulomb failure criterium, and the reinforcement as an elastic membrane having a certain tension strength. The procedure described in the next chapter will first be verified by its application to the data of some model tests and then used for predicting the effects of the surface reinforcement of soft soils under a road embankment.

In the present phase of our investigations the viscous properties of materials will be neglected. The consolidation which is governed by the hydraulic seepage resistance will be considered only by separate analyses of the reinforcing effect in the beginning phase of pure distorsional deformations and in the phase at the end of primary consolidation. The effect of the number of reinforcing layers will be elucidated as well.

## NUMERICAL PROCEDURE

The computer program for the numerical analysis according to the finite element method has been elaborated for plane strain states. The contour of the compatible network of finite elements can be arbitrary provided that it is composed of straight lines between external nodes of triangular elements into which the soil body has been discretized; the network can include the embankment and several soil layers. The triangular elements have six degrees of freedom. The reinforcement is continuously composed of linear elements with four degrees of freedom; their nodes coincide compatibly with the nodes of the finite element mesh of the soil body. On the contour of the soil body either nodal forces or nodal displacements or partly forces and partly displacements can be prescribed.

The hypoelastic stress-strain relationships for soils are expressed as non-linear relationships between octahedral values of strains ($\varepsilon^o$ , $\gamma^o$) and stresses ($\sigma^o$ , $\tau^o$):

$$\varepsilon^o = \varepsilon^o(\sigma^o, \tau^o) \quad , \quad \gamma^o = \gamma^o(\tau^o, \sigma^o) \qquad (1), (2)$$

either by analytical functions or by sets of coordinate values using splines for interpolation. Equation (2) includes the failure states at which the shear strains tend to infinity. Equations (1) and (2) yield the corresponding relationships between the tangent values of the compression ($K$) and shear ($G$) moduli resp. and the octahedral stresses:

$$K = K(\sigma^o, \tau^o) \quad , \quad G = G(\tau^o, \sigma^o) \qquad (3), (4)$$

whereby

$$K = (1/3)/(\partial\varepsilon^o/\partial\sigma^o), \quad G = 1/(\partial\gamma^o/\partial\tau^o) \qquad (5), (6)$$

In soils the strains are deduced from displacements according to the first order theory.

For the reinforcement linear stress-strain relationship has been taken, the tension strength representing the limit of the elastic behaviour. According to the membrane character of the reinforcement, the second order theory has been applied.

Neither consolidation nor viscous properties have been considered in the present phase of our analysis.

## MODEL TESTS

The reinforcement effect has been illustrated by some tests in a model kit 28 cm high, 29.7 cm wide and 150 cm long. Into this kit a remoulded saturated silty clay of very high compressibility was placed in layers of 3 cm up to the height of 18 cm ; the upper surface of each layer was colored. The surface of the highest layer was covered by a 3 cm thick layer of fine sand. A rigid wooden plate $b$ = 29 cm wide and $l$ = 9.5 cm long was put onto the sand in the middle of the kit. The average pressure below the plate was first 3.5 kPa and increased in time intervals of 24 hours by 7 kPa .

Two test series have been performed: the first without the fabric, the second with the fabric put in a length $3\,l$ = 28.5 cm on the clay surface before covering it with sand. The product of the modulus of elasticity $E_r$ and the thickness $a$ of the fabric was $a\,E_r$ = 1350 kN/m . The water content of the clay was determined at the placing ($w_{in}$) and at the end ($w_{end}$) of each test series. The following values were ascertained (1st , 2nd series):

$w_{in}$ = 35.7 , 44.3 % , $w_{end}$ = 44.7 , 44.6 % .

On one of both longitudinal lateral plexiglass plates of the kit a network of vertical and horizontal lines was marked in order to facilitate the registration of the displacements of coulored interlayer surfaces. In both test series the displacements were observed up to the failure load reaching the values of $q_f$ = 24.5 and 38.5 kPa respectively.

Since the model scales cannot be successfully realized, the model test results cannot be directly applied for prediction of the behaviour of a prototype. Nevertheless, the comparison of measured displacements with computed values obtained by using the material properties and data of model tests, can serve for the verification of the suitability of the numerical procedure. In our concrete case the following assumptions could be taken: (1) the displacements developed in undrained conditions (Poisson´s ratio close to 0.50), (2) the model soil can be considered as an ideal elastic-plastic body with deformation modulus $E_s$ obtained in the domain of approximately linearly increasing settlements ($q$ = 10.5 kPa) of the unreinforced soil, and with the cohesion $c$ (at $\phi$ = 0) according to Prandtl´s solution and corresponding to the load $q$ at which the settlements started to increase with acceleration ($q$ = 24.5 kPa). - Since the

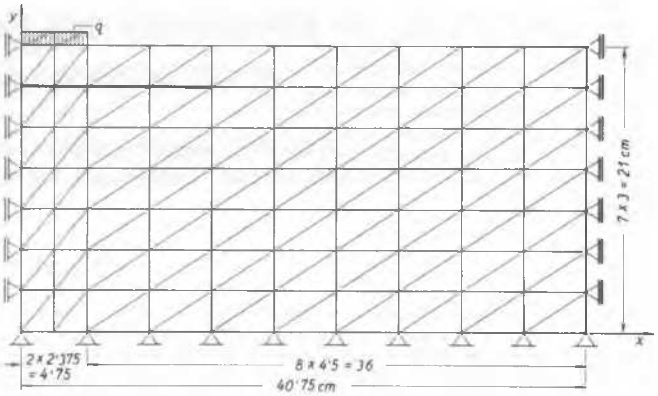


Fig.1 Model test: network of finite elements and boundary conditions.

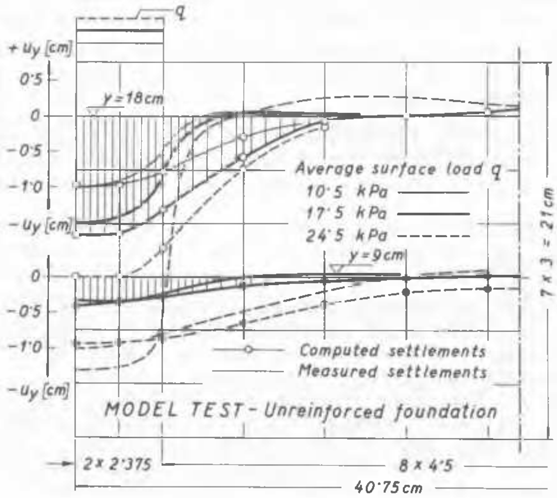


Fig.2 Model test: without fabric. Comparison of computed and observed settlement diagrams.

water content and with it the deformability could not be kept quite constant in successive test series, the $E_s$ modulus in the second series (reinforced footing) was deduced from the settlements in the load axis at $q$ = 17.5 kPa . The following values were obtained:

$E_{s1}$ = 42 kPa , $E_{s2}$ = 65 kPa (both values at $\nu$ = 0.48), $c_1 = c_2$ = 4.8 kPa .

The comparison of measured settlements and those computed with $E_{s1}$ for the first series (unreinforced soil) and $E_{s2}$ resp. for the second series (reinforced soil with $a\,E_r$ = 1350 kN/m), with $c$ = 4.8 kPa , $\nu$ = 0.48 for both series, is presented by diagrams $u_y = \{u_y(x)\}_{y=18\text{ cm}}$ in Figs 2 and 3 resp. while Fig.1 presents the finite element mesh used and the boundary conditions; the rigid loading plate was discretized in four triangular elements of very high stiffness ($E_q = 5\,.\,10^8$ kPa at $\nu_q$ = 0). For both series the settlement diagrams are presented for three load intensities: $q$ = 10.5 , 17.5 and 24.5 kPa . In the second test series (Fig.3) (reinforced soil) the computed settlements approach satisfactorily the measured values. The discrepancies in the first series (without fabric) are probably mainly due to the effect of the penetration of the rigid plate through the sandy covering layer (confer Tcheng, 1957). In greater depths the difference between computed and measured values is not important even in the unreinforced soil. (For the level $y$ = 9 cm the comparison is presented in Fig.2.)

The local safety has been defined by the quotient $\tau^o_f/\tau^o$ , $\tau^o$ being the octahedral shear stress and $\tau^o_f$ its failure value: $\tau^o_f = (2\sqrt{2}/3)\ c$ (confer equation 9 for $\phi = 0$). For the load $q$ = 17.5 kPa the isolines $\tau^o_f/\tau^o$ are presented in Fig.4 for the unreinforced soil and in Fig.5 for the reinforced soil. The favourable effect of the reinforcement on the increase of the bearing capacity is evident.

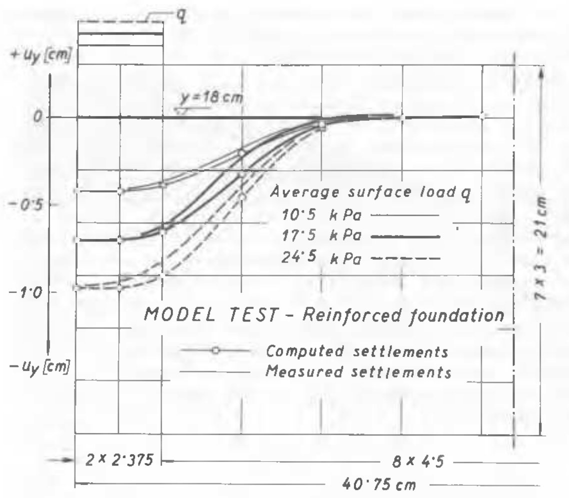


Fig.3 Model test: with fabric. Comparison of computed and observed settlements.

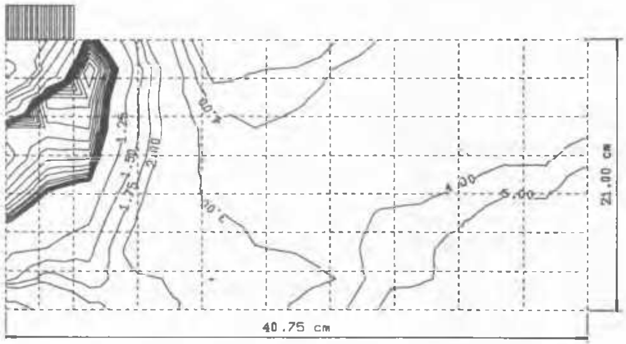


Fig.4 Model test: without fabric. Isolines $\tau^o_f/\tau^o$ at $q$ = 17.5 kPa .

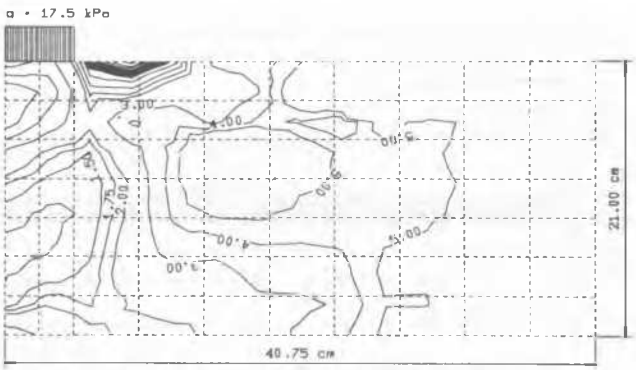


Fig.5 Model test: with fabric. Isolines $\tau^o_f/\tau^o$ at $q$ = 17.5 kPa .

In general the above comparisons confirm the applicability of the elaborated computer program.

APPLICATION TO A ROAD EMBANKMENT

The above described and verified computer program was applied to a symmetric trapezoidal road embankment (width 9 + 2 x 9 = 27 m) put directly on a very compressible saturated silty clay of 9 m thickness lying on a horizontal rigid base. The tangent compression ($K$) and shear ($G$) moduli of the clay could be expressed by the following analytical functions:

$$K = a + b\ \sigma^{o\prime} \quad (\sigma^{o\prime} \text{ positive for tension}) \qquad (7)$$

$$G = f(\tau^o_f - \tau^o) \qquad (8)$$

with the following parameter values:

$a$ = 100 kPa , $b$ = - 8 , $f$ = 5.27

$\tau^o_f$ is the octahedral shear stress at failure:

$$\tau^o_f = \{2\sqrt{2}\cos\phi'/(3 - \sin\phi')\}(c - \sigma^{o\prime}\tan\phi') \qquad (9)$$

$c$ = 7 kPa , $\phi' = 25^o$

The initial stress state corresponds to the void ratio $e_0$ = 2.92 in stressless state and to the earth-pressure coefficient at rest $K_0$ = 0.667 . The discretization of the soil layer into a network of triangular finite elements and the boundary displacement conditions are shown in Fig.6 . The discretization is denser near the surface between the depths 0.30 , 0.60 and 0.90 m representing the levels of the triple reinforcement by non-woven geotextile with the value $a\ E_r$ = 4350 kN/m . The single reinforcement by the same membrane was placed in the depth 0.60 m and the double one on the levels 0.30 and 0.90 m . The unit weight of the embankment fill has been taken 20 kN/m$^3$ ; the embankment profile was not included into the network of finite elements; its weight has been considered to be transferred on the soil by surface nodal forces linearly proportional to the height of the fill above the nodes. The reinforcements have been laid along the entire fill-soil contact and on both sides by 9 m outside the foot of the embankment.

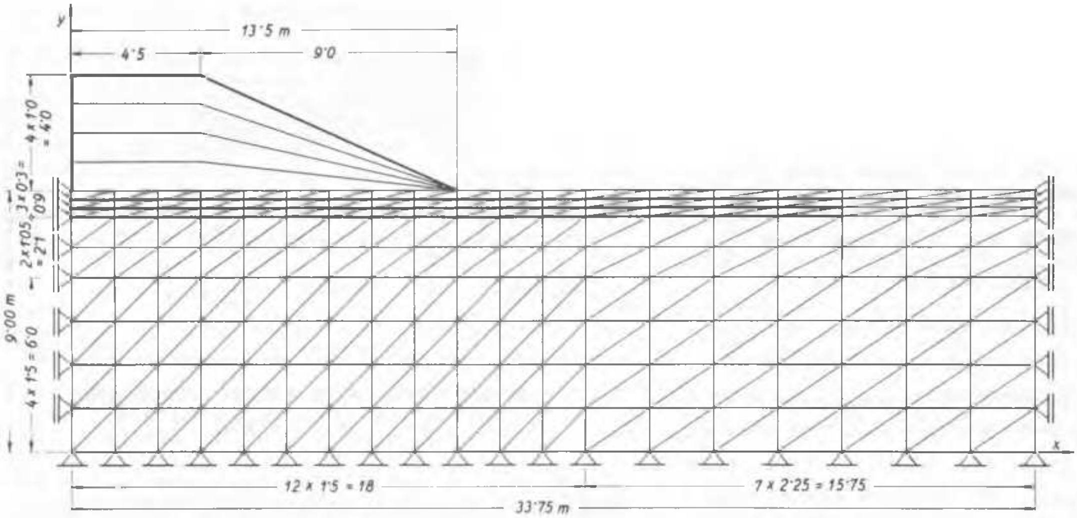


Fig.6 Road embankment. Network of finite elements and boundary conditions.

With the above defined material properties , displacements, strains and stresses in the soil have been computed for several embankment heights up to 6 m . For the heights 1 , 2 and 4 m the resulting settlement diagrams $u_y = \{u_y(x)\}_{y=9\text{ m}}$ are presented in Fig.7 ; two alternatives have been considered: the unreinforced soil and the triply reinforced soil. For heights 1 and 2 m the reduction of settlements by reinforcement is not important and could hardly justify the costs of reinforcement. The settlements of the 4 m high embankment are too large to be tolerated; neither do they allow the application of the first order theory. (The program can, however, easily be extended to allow for large deformations, similarly as by Boutroup and Holtz, 1983.)

Now, the above presented "drained" analysis can give a definite answer concerning the safety of the embankment and the role of the reinforcement only in cases when the consolidation is forwarded by vertical drainage, event. combined with preloading, and on condition the loading proceeds slowly enough. Even in such cases the speed of the pore-pressure dissipation has to be taken into account at least by an approximate analysis. For undrained and non-preloaded soils a rough evaluation of displacements and safety can be made by combining the "drained" and "undrained" analysis. For the latter the total stress-strain relationships have to be applied as deduced from undrained triaxial or biaxial tests. Since the undrained displacements occur at unchanged porosity and since during the consolidation the spheric deformations are accompanied by additional deviatoric displacements, the final settlements approach the values obtained by superposing undrained and drained displacements.

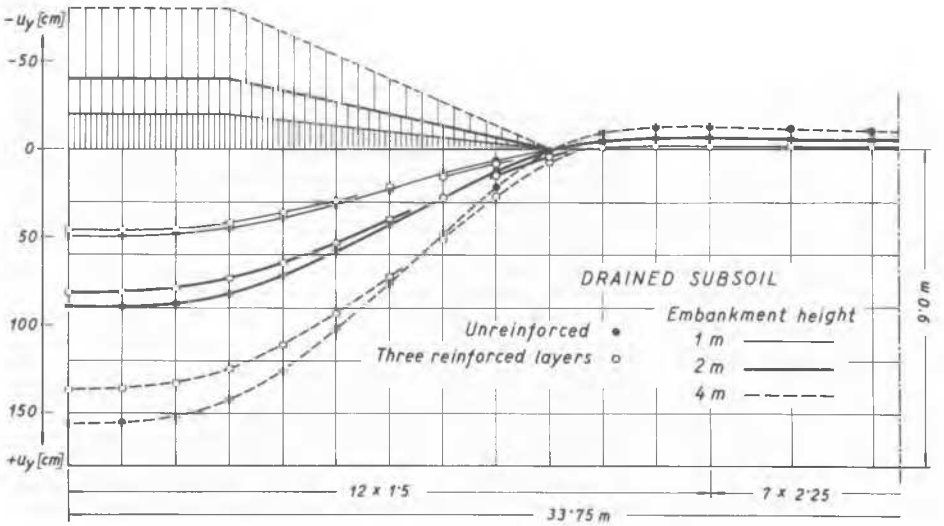


Fig.7 Road embankment: drained conditions. Comparison of settlements of unreinforced and of triply reinforced soil surface: for three different embankment heights.

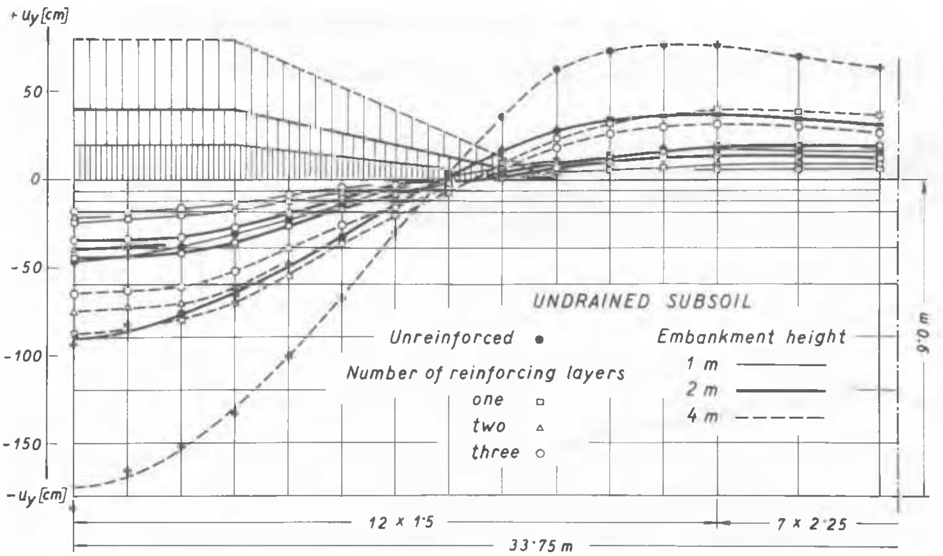


Fig.8 Road embankment: undrained conditions. Comparison of settlements for three embankment heights (1 , 2 and 4 m) and for four reinforcement alternatives: unreinforced soil, single, double and triple reinforcement.

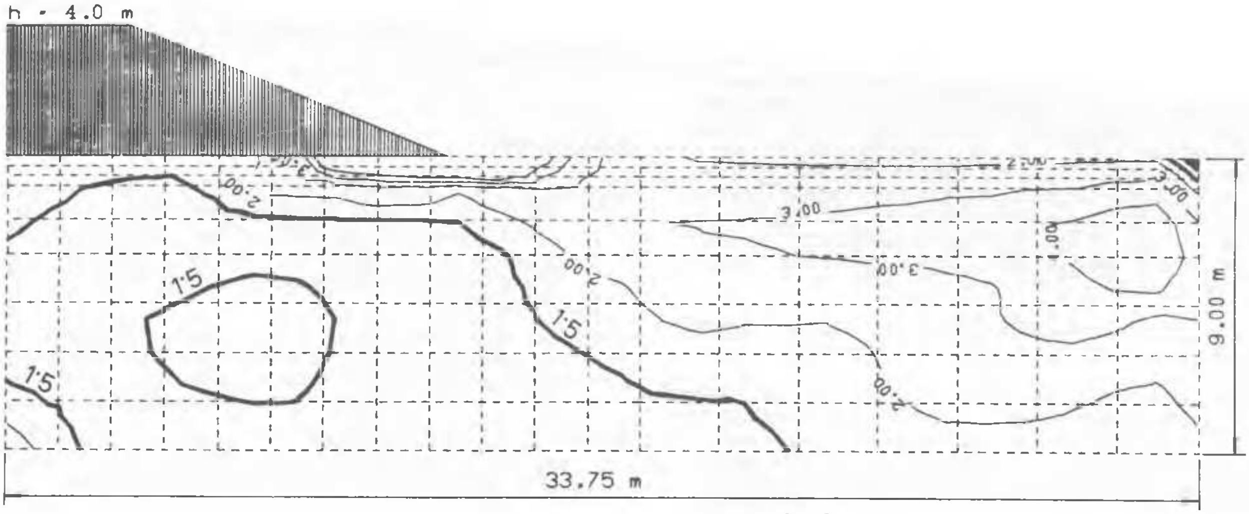


Fig.9 Road embankment: undrained conditions. Isolines $\tau_f^o/\tau^o$ for the unreinforced soil at the embankment height 4 m .

Missing analytical expressions for the undrained behaviour of the same soil we have repeated the above presented "drained" analysis with unchanged $G = G(\tau^o, \sigma^o)$ relationship at $\nu = 0.499$ , reducing the strength parameters to $c = 5$ kPa , $\phi = 12^o$ . The resulting settlement diagrams are presented in Fig.8 for the unreinforced as well as for all three reinforced alternatives (one, two and three reinforcement layers). The diagrams prove favourable reinforcement effect up to the embankment height 4 m with modest yet pronounced decrease of settlements with increasing number of reinforcement layers.

For the embankment height 4 m , Fig.9 represents isolines $\tau_f^o/\tau^o$ ($\tau_f^o$ according to equation 9) for the unreinforced soil, and Fig.10 for the single reinforcement. The comparison proves the favourable effect of reinforcement on the increase of safety.

DISCUSSION AND CONCLUSIONS

Additionally the described computer program was applied to the case treated in the foregoing chapter with the only difference that down to the depth 0.90 m below the fill-soil contact the silty clay was replaced by a fine sand with a constant deformation modulus $E_c = 20\ 000$ kPa at $\nu = 0.3$ . Already at the height 1 m of the embankment plastification ($\tau_f^o/\tau^o < 1$) began to

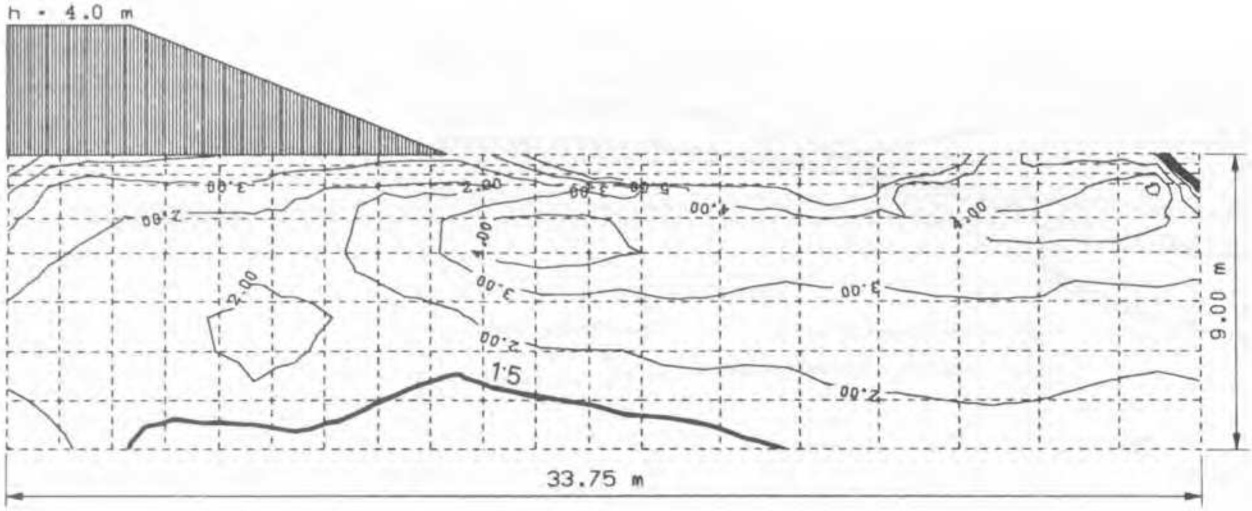


Fig.10 Road embankment: undrained conditions. Isolines $\tau_f^{o}/\tau^{o}$ in the case of single reinforcement, height 4 m .

develop in the sandy cover, provoked probably by large horizontal displacements of the clay in the contact surface. In the program of further investigations we intend to introduce special interface elements between the fabric and adjacent soil as well as along contacts between soils of very different deformation properties allowing for slip along interfaces. Such interface elements have already been developed by Barksdale & all., 1983, while Brown and Poulos, 1981, take into consideration the slip along the soil-reinforcement interfaces only after full mobilization of friction; constant resistance is assumed thereafter (see also Mitchell and Katti, 1981).

With the above indicated limitations the present study permits the following conclusions:

(1) The geotextile membrane restrains the lateral displacements of the subsoil, favourably modifies the displacement pattern reducing somewhat maximum settlements and increases the bearing capacity (in accordance with Schlosser & all., 1983).

(2) In poorly permeable undrained soils the reinforcing effect is particularly pronounced in the initial consolidation phase of prevailing distortional displacements.

(3) The finite element method is a useful tool for predicting the reinforcing effect.

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