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NUMERICAL ANALYSIS OF REINFORCED COHESIVE SOIL FILLED EMBANKMENTS ON SOFT SOIL

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ABSTRACT: The performance of embankments resting on soft soil subgrades is of practical interest to all the engineers. Soil reinforcement techniques have come in to practice to improve the performance of these structures. With the recent development of sandwich technique (so called because the reinforcement is sandwiched between two strong soil layers), it is possible to use locally available materials for the construction of these structures. This paper discusses the finite element techniques that can be adopted for the analysis of such structures and an advanced constitutive models for simulating the performance of soils. The paper will also present several results obtained from parametric finite element analysis of reinforced soil embankments constructed of clay soils using sandwich technique.

1. INTRODUCTION

Embankments are required to support many engineering facilities such as railway lines and motor ways. When these are constructed for transportation purposes, they will have to pass through areas of compressible soil such as coastal regions or estuaries of rivers. Under such circumstances, it was a practice to select appropriate sites having strong subgrades for constructing these structures. However, in many cases it may not be possible to choose the sites and the structures have to be constructed even on poor subgrades. The low strength and high compressibility of soft subgrades result in large deformation and lower stability of embankments.

Reinforced soil technique can be used effectively to impart additional strength to soils. In reinforced soil, the reinforcement imparts additional strength by way of stress transfer through the interfacial friction. This necessitates the use of high strength frictional backfill material. However, replacing the soft soil and using good quality imported backfill soil may not be economically feasible. In order to overcome this problem, a technique has recently been developed in which even poor quality soils (cohesive or soils with low friction angle) can be used as backfills (1,2,3). This technique, referred to as sandwich technique, involves in providing thin layers of high frictional soil above and below the reinforcement. The rationale behind this concept is that most of the stress transfer from soil to reinforcement happens within a thin layer around the reinforcement as demonstrated experimentally, (4). Using this sandwich technique it is possible to use the locally available soil as backfill soil resulting in substantial savings in project costs. In addition, the sandwich layers function as drainage media also, thus preventing the build up of pore-pressures within the embankment.

This paper investigates the improvement in the performance of reinforced soil embankments with cohesive soil fill resting on soft foundations due to the introduction of sandwich technique. A Hierarchical Single Surface (HiSS) Constitutive Model with appropriate modifications has been used to model the behaviour of cohesive soil. This constitutive model is capable of predicting the behaviour of soil under various stress paths. Results from a detailed parametric investigation that show the effect of various parameters on the behaviour of these embankments are presented in this paper.

2. FINITE ELEMENT MODELLING

The reinforced soil structure consists of three major constituents, namely, backfill soil, reinforcement layers and interfaces between the soil and reinforcement layers. This system was modelled using discrete finite element technique in which all the three constituents are considered separately. Using this approach, the strains developed within the reinforcement layers can be predicted and the actual load-transfer mechanism from soil to reinforcement can be simulated rigorously.

The incremental finite element equilibrium equations considered are of the type shown in Eq. 1 in which the load vector is expressed as the difference between the external load vector and the internal reaction force vector computed from the element stresses of previous iteration.

$$[K]\{\Delta u_i\} = \{P\}_{exti} - \Sigma[B]^T \{\sigma_{i-1}\} \qquad (1)$$

This analysis scheme allows for carrying forward any error in the out-of-balance force to the next iteration (or next load step) thus satisfying the global equilibrium at all the load steps. The finite element solution was iterated until the out-of-balance force norm is less than 0.5%. Typically about 6 iterations were performed at each load step.

3. HIERARCHICAL SINGLE SURFACE (HiSS) CONSTITUTIVE MODEL

The soil in different parts of the embankment undergoes loading in different stress paths during the gradual construction of embankments, e.g. pure shear, direct extension, compression etc. The strength of soil is different under different stress paths. It is essential that this difference in behaviour is well accounted for in the constitutive models for soils.

The behaviour of soil in the embankment is simulated using Hierarchical Single Surface (HiSS) model, which is capable of simulating the behaviour of soil under different stress paths. The major advantage of this model is that all the parameters can be determined using data from simple triaxial and hydrostatic compression tests. The details of this model can be found in [5]. The model reported in that paper is valid for normally consolidated clays only which do not have cohesive strength and are referred to as δ* models. This model has been extended in this investigation to analyse soils that have both cohesive and frictional strength. The constitutive model is briefly described below.

Yield Surface, $F = (J_{2D} / P_a^2) - F_b F_s = 0$ (2)

P_a is the atmospheric pressure and $F_b = -\alpha_{ps} (J_1/P_a)^n + \gamma(J_1/P_a)^2$ (3)

in which α_{ps} is the hardening or growth function that controls the movement of the yield surface; the function F_b describes the shape of F in $J_1 - \sqrt{J_{2d}}$ space; and γ and n are material parameters. The parameter n defines the phase change point, where material changes from contractive to dilative behaviour. In the case of normally consolidated cohesive soils, transition takes place at the critical state line and hence the phase change line coincides with the critical state line. Phase change line connects the crest of all the yield surfaces. The function F_s describes the shape of F in the octahedral plane and is given by

$F_s = (1 - ß\, S_r)^m$ (4)

where S_r is a stress ratio, and here
$S_r = (\sqrt{27}/2)\, J_{3d}\, J_{2d}^{-3/2}$ (5)

J_1 is the first invariant of stress tensor. J_{2d} and J_{3d} are the second and third invariants of deviatoric stress tensors, ß and m are material parameters.

The hardening function is a dependent on various internal variables related to plastic deformations. In this model, the hardening function is given by

$$\alpha_{ps} = \frac{h_1}{(\xi_v + h_3\,\xi_D^{h_4})^{h_2}} \tag{6}$$

where h_1, h_2, h_3 and h_4 are material parameters. The increments of trajectories of volumetric and deviatoric plastic strains are defined as follows.

$$d\xi_D = (d\,\epsilon_{ij}^P\;d\,\epsilon_{ij}^P)^{1/2} \tag{7}$$

$$d\xi_V = \left(\frac{1}{\sqrt{3}}\right)\left|d\varepsilon_V^P\right| \tag{8}$$

where $d\,\epsilon_{ij}^P\ (= d\varepsilon_{ij}^P - \delta_{ij}\,d\varepsilon_V^P/3)$ is the incremental deviatoric plastic strain tensor and $d\varepsilon_V^P$ is the incremental volumetric plastic strain due to virgin loading. The model treats loading, unloading and reloading up to the point of unloading separately. Various constitutive parameters can be obtained from simple laboratory tests.

4. FINITE ELEMENT PARAMETRIC ANALYSES

The embankment was analysed as a plane strain case by taking a unit slice of a long embankment. Only half of the embankment is analysed by using the symmetry boundary conditions along the mid-section of embankment, i.e. the lateral deformations are set to zero along the vertical axis at the mid-section of embankment. The soil was simulated using three node isoparametric continuum elements with one point integration. Four 3-node triangles were placed within each triangle as illustrated in Figure 1. This mesh arrangement was shown to give accurate results with lower computational efforts for all plane strain problems, [6]. The constitutive behaviour of soil was simulated using the HiSS model described above. The excess stresses beyond the yield surface are corrected along the flow direction by using a modified correction scheme. The backfill was taken as a cohesive soil whereas the soil in the sandwich layer was taken as a granular soil. The soil in the foundation was assumed to have low shear strength properties and low stiffness to simulate the soft subgrades.

The reinforcement was modelled using two node isoparametric prismatic elements. As the geosynthetic reinforcements are flexible and cannot sustain compressive loads, the compressive forces were not allowed to develop within the reinforcement elements. The reinforcements made of synthetic and metallic materials has been considered in the analysis by using wide range of modulus values for the reinforcement. Although several reinforcement stiffnesses were considered in the analysis, only the results for J=2000 kN/m are presented in this paper due to lack of space.

The interfaces between various materials were modelled using four node elements of the type developed in [7]. The order of displacement variation in these elements is compatible with that in the three-node interfaces. The stresses in these elements were computed as a function of the relative displacements between the two surfaces on each side of the interface. The stiffness of these elements is formulated in terms of two independent stiffness values, one in the tangential (shear) direction and the other in the normal direction. Both the stiffness components were assigned very high values to enforce displacement compatibility at the interface until the onset of failure defined by Mohr-Coulomb failure law. When the shear stress exceeds the shear strength, the shear stiffness was set to a small value to allow for relative movement to take place. The normal stiffness was reduced to a small value under tensile normal stresses to allow for debonding to take place. The shear strength of the interfaces was assumed to be equal to that of the surrounding soil.

The relevant constitutive parameters of a typical soft clay (occurring around Chennai city) determined from simple laboratory tests are given in **Table 1**. The properties of the sand used in sandwich layers are given in **Table 2**. The properties of reinforcement and interface are given in **Tables 3** and **4** respectively. In the investigation, three different foundation depths were considered.

Table 1 : Properties of soil

Material	Fill	Foundation
Initial modulus	25820 kPa	10000 kPa
Density	16.7 kN/m^3	16.7 kN/m^3
γ	0.0621	0.062
n	2.57	2.5
h_1	0.0028	0.0025
h_2	1.48	1.44
h_3, h_4, β	0	0
m	-0.5	-0.5
J_{2Di}	2106	800

Table 2: Properties of sand layer

Initial modulus	30000 kPa
Poisson's ratio	0.35
Cohesion	0
Internal friction angle	42°
Dilation angle	10°
Hardening parameter	0
Density	20 kN/m^3

Table 3: Properties of interface

Material model	Linear elastic with Mohr-Coulomb failure limit	
Interface with	Clay soil	Sand
Initial tangential stiffness	10^6 kN/m^3	10^6 kN/m^3
Initial normal stiffness	10^6 kN/m^3	10^6 kN/m^3
Residual tangential stiffness	100 kN/m^3	100 kN/m^3
Residual normal stiffness	10^4 kN/m^3	10^4 kN/m^3
Cohesive strength	36 kPa	0
Interface friction angle	0°	42°

Table 4: Properties of reinforcement

Material model	Strain dependent material
Reinforcement stiffness (J)	2000 kN/m

compressive forces were not allowed to develop within the reinforcement elements.

A crest width of 6 m and a height of 3 m were considered for the embankment fill. Only half the section was considered for analysis due to the imposition of symmetry boundary conditions. Although several side slopes were considered in the investigation, only the results for a side slope of 2:1 (H:V) are presented here due to lack of space. Three depths of the soft subgrade (D), viz. 3, 6 and 15 m were considered in the analysis. A single layer of reinforcement was considered at the fill-reinforcement interface. A typical mesh used for the analysis is shown in **Figure 1**. The displacements were monitored at three different locations (all lying on the surface of subgrade soil), one point at the mid-section of the embankment, one at the crest position and the other at the end of slope. Similarly, element stresses and strains were also monitored at three different locations on the soft ground surface. Surcharge loading of 800 kPa was applied in 400 load steps on the crest to simulate gradual increase in the height of the embankment.

5. RESULTS AND DISCUSSION

The finite element results showed that the vertical and lateral deformations are reduced with the provision of sand layers around the reinforcement. The reduction in deformations for various points on the ground surface for the embankment with 3 m deep foundation are shown in **Figures 2** and **3**. The horizontal and vertical deformations are normalised with respect to the base width of embankment (W) and depth of soft foundation (D).

A comparison of the reinforcement forces developed with and without the use of sandwich layers is shown in **Figure 4** (D=3 m). It is clear that the sandwich layer helps in better load transfer from the soil into the

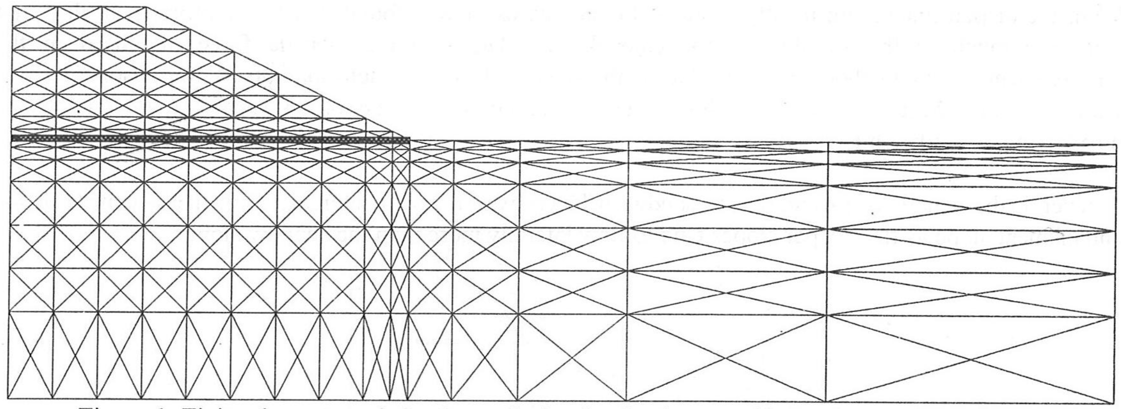

Figure 1. Finite element mesh for the analysis of embankment with 6m deep soft foundation

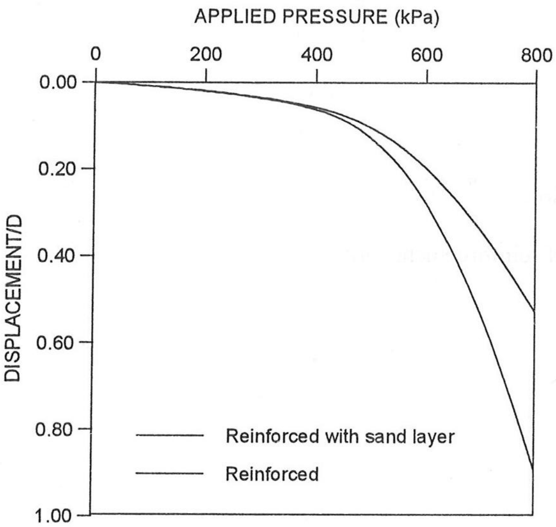

Figure 2 Normalised vertical displacement of the point on the ground at the symmetry axis

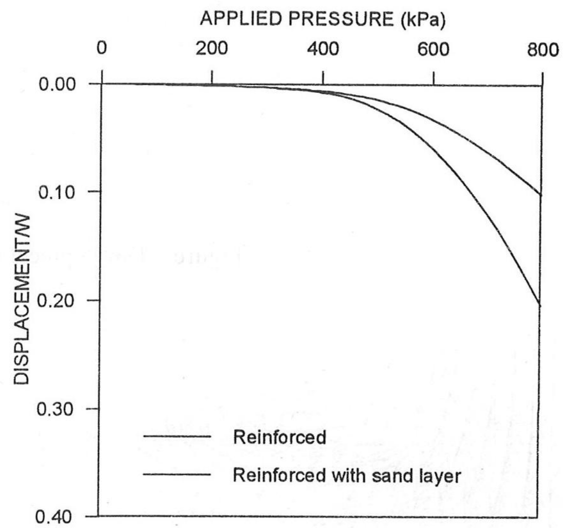

Figure 3 Normalised horizontal displacement of the toe of the embankment

reinforcement layer. In the other case, the reinforcement layer pulls out of the soil because of low interfacial shear strength thus it will not be able to develop higher forces.

The largest reduction of 41% in vertical settlements due to the introduction of sandwich layers was obtained at the mid-section of embankment for a subgrade depth of 3 m. For foundation depths of 6 and 15 m, the respective reductions in vertical settlements were 34% and 30%. Comparatively, higher reductions were obtained in the lateral deformations due to the provision of sandwich layer. The reduction in overall deformations can be noticed from the plot of displacement vectors of reinforced embankment with and without sandwich layer (**Figures 5** and **6**).

In contrast, increased reinforcement strength mobilisation for the same surcharge load was noticed with increasing depth of foundation. The total deformations (vertical and horizontal) of the embankment increase as the depth of soft soil subgrade increases thus developing higher reinforcement forces. For 3 m depth foundation, the deformations were mostly horizontal. The horizontal deformations lead to an increased mobilisation of interface stresses, which leads to a reduction in horizontal deformations. At the same time, the horizontal pull of the reinforcement effected rigid support action of the reinforcement and the sand layer for the superimposed load. This is evident from the comparison of vertical stress contour diagrams shown in **Figures 7** and **8** (6 m deep foundation). In the case of embankments resting on soft subgrades of large depth, the displacements are mostly vertical. In such cases, it was found that the reinforcement does not contribute much to the stability of the embankment. The additional tensile force mobilised in the reinforcement layers in those cases is due to the increased vertical deformations. Comparison of shear strain contours (**Figure** 9 and **10**) shows that the remarkable reduction in shear strains due to the introduction of sandwich technique.

In general, the benefit of providing the sandwich layers was found to increase with the steepness of the embankment slope as the steeper slopes tend to flow laterally more than the shallow slopes.

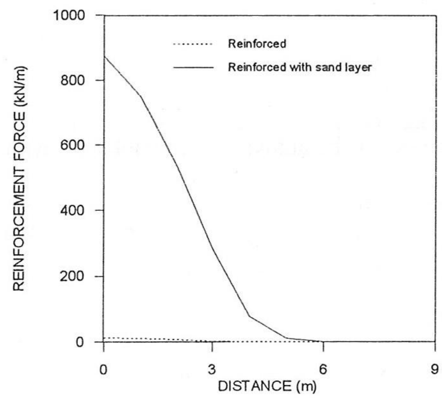

Figure 4 Development of reinforcement force

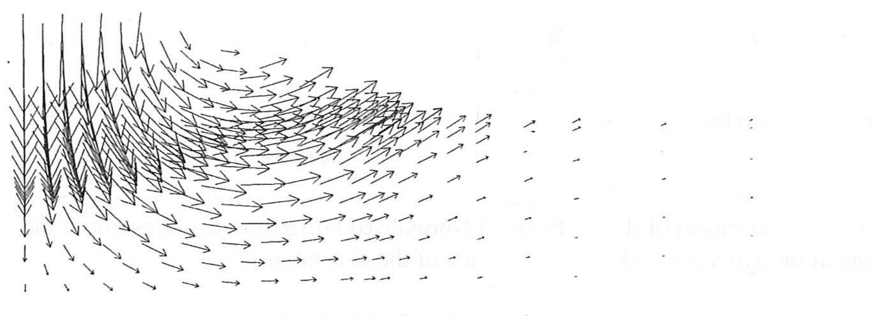

Figure 5. Displacement vectors for reinforced embankment without sandwich layer

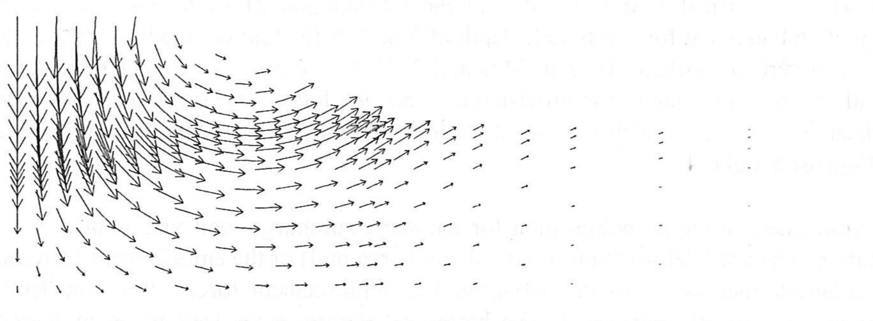

Figure 6 Displacement vectors for reinforced embankment with sandwich layer

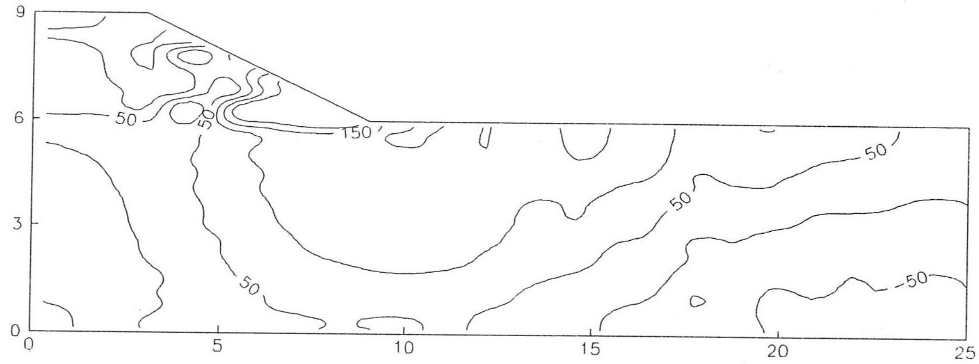

Figure 7 Vertical stress contours for reinforced embankment with no sandwich layer

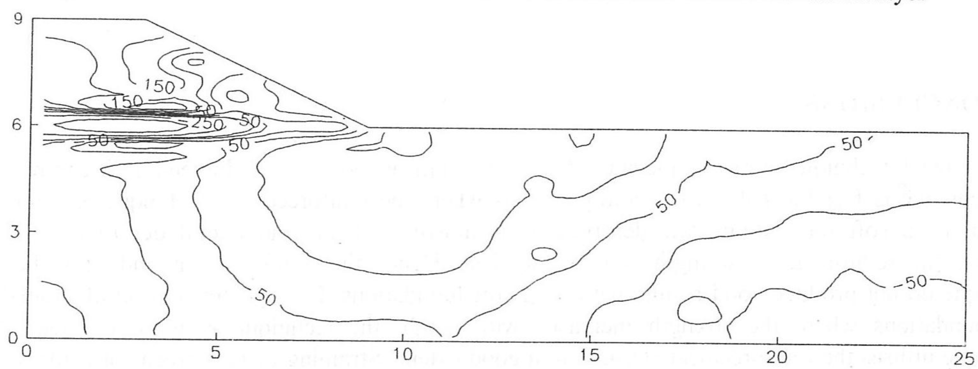

Figure 8 Vertical stress contours for reinforced embankment with sandwich layer

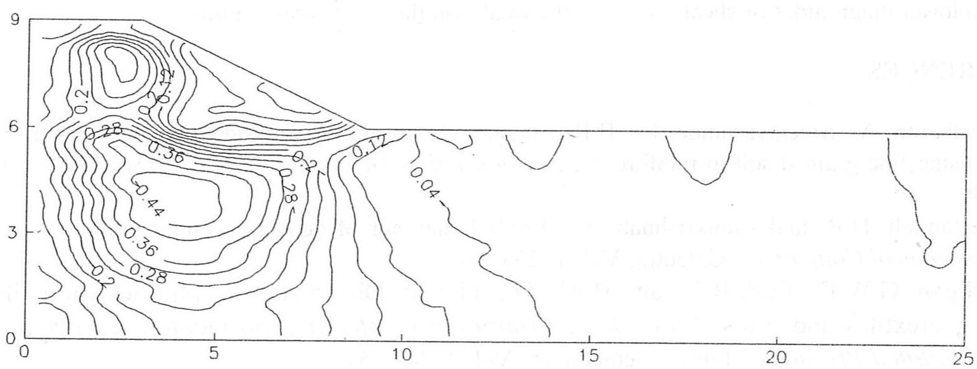

Figure 9 Shear strain contours for reinforced embankment with no sandwich layer

Some analyses were performed by providing an additional reinforcement layer at some depth below the ground level. The performance of these embankments was found to improve substantially due to a drastic lateral confinement of internal reinforcement layer. Maximum benefit of this additional reinforcement can be obtained by locating it at a depth where the highest lateral soil strains occur. This depth was found to be dependent on the angle of the slope, crest width and height of embankment and the depth of foundation soil.

Results obtained from repeated load analyses (cyclic loading) showed that the sandwich layers slow down the accumulation of strains and the structures can be subjected to more number of cycles than the conventional embankments. The relative improvement of sandwich layers was found to be more in the case of cyclic loading than under monotonic loading.

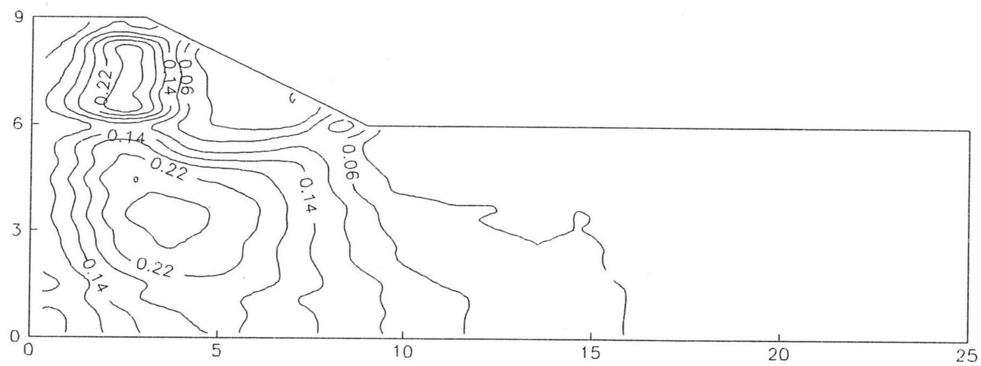

Figure 10 Shear strain contours for reinforced embankment with sandwich layer

6. CONCLUSIONS

The sandwich technique improves the stability of embankments on soft soil. In general, it can be stated that the technique is feasible only in the configurations where the reinforced soil technique is suitable. In the case of deep soft foundations, the deformation is mostly vertical, and lateral deformation required to mobilise the reinforcement strength is relatively less. Hence the reinforcement and thus the sandwich technique do not produce good results with deep soft foundations. For soft foundation of limited depth or for foundations where the strength increases with depth, the technique is more efficient. Sandwich technique utilises the reinforcement strength to a good extent. Straining of reinforcement leads to increased magnitudes of stresses near the interface which requires high strength soil at the interface. Thus the modulus as well as the interfacial friction angle of the sand are utilised. Reduction in overall deformations leads to lower magnitudes of shear strains in the weak soil thus preventing failure.

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