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On the Design of Reinforced Embankments on Soft Brittle Clays

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ABSTRACT

Finite element analyses are used to examine the effect of geosynthetic modulus on the behaviour of reinforced embankments constructed on very soft brittle clay deposits with and without a higher strength surface crust. Results are presented which illustrate how the calculated shear strains in the very soft underlying foundation are influenced by the geosynthetic modulus and selection of a limiting strain in design. It is demonstrated how an examination of shear strain in the foundation soil may be used to assess the potential for problems if the soil is susceptible to strain softening. Implications for practical design are discussed.

INTRODUCTION

It is now well recognized that the modulus of geosynthetic reinforcement greatly influences the performance of embankments constructed on soft cohesive deposits. The purpose of this paper is to discuss the effects of reinforcement modulus on the development of strain in both the geosynthetic and the underlying foundation soil, with emphasis on very soft brittle cohesive soils. The type of foundation soils being considered are typical of soft elastic-perfectly plastic soils with and without a higher strength surface crust.

Results obtained from carefully performed finite element analyses are used to illustrate that potential problems may arise due to strain softening in some brittle soils as a result of adopting allowable reinforcement strains recommended in the literature in a design situation. Some recommendations are made relating to the design approach which should be adopted for these situations.

NUMERICAL MODEL

Results from small strain elasto-plastic finite element analyses are presented in this paper. The program used was a modified version of program AFENA, developed by Carter (1985). Specific details regarding the formulation are discussed by Rowe and Soderman (1987) and Rowe and Mylleville (1988).

The results presented here were obtained using a finite element mesh with 12171 degrees of freedom. The high degree of refinement of the finite element mesh required the use of an ETA10-P supercomputer to perform the analyses.

Reinforced embankments were "constructed" by turning on gravity within rows of elements. This involved up to 12 lifts and a total of up to 250 load steps in the analysis.

PROBLEM DESCRIPTION

The type of problem being considered is that of a geosynthetic reinforced embankment constructed on a soft clay foundation as shown in Figure 1. Embankments of crest width B=18 m, were "constructed" on a soft clay deposit to a height h above original ground level, using 2 to 1 side slopes.

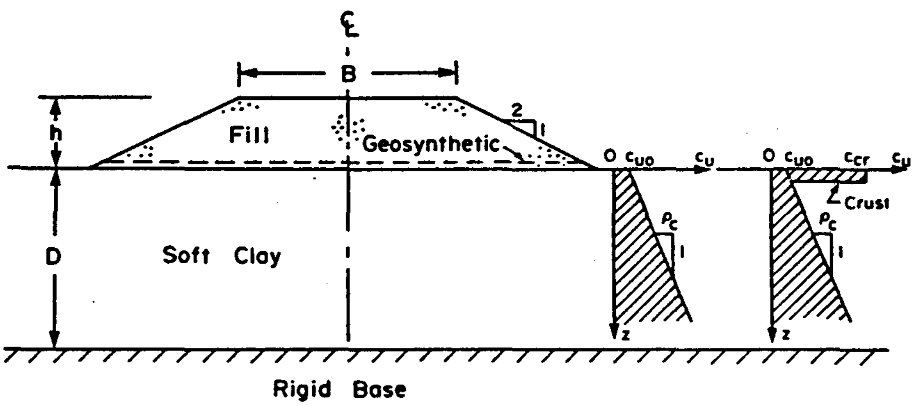

Figure 1. Typical embankment on a soft clay foundation

The granular embankment fill was modelled as having a stress-dependent Young's modulus based on Janbu's equation (see Rowe and Soderman (1987)). The parameters used in the numerical model for the granular fill were ϕ=32°, γ=20 kN/m^3, Poisson's ratio, ν=0.35 and dilatancy, ψ=0°.

The depth of clay deposit D, being considered is 15 m which is in turn underlain by a rigid base as shown in Figure 1. When constructing embankments on soft clay deposits, the most critical stage in terms of stability usually corresponds to that at the end of construction. The finite element results presented herein are predictions of embankment behaviour based on short term undrained (ϕ=0) analyses (taking Poisson's ratio to be 0.48).

The soft foundation soil was modelled as having an undrained shear strength and undrained modulus which increase linearly with depth from some surface value as shown in Figure 1. Some results are presented for undrained strength profiles with and without a higher strength surface crust. Strength profiles such as those shown in Figure 1 are commonly encountered in soft, normally or slightly overconsolidated clays. The ratio of undrained modulus to undrained strength (E_u/c_u) considered was E_u/c_u=500. The clay foundation was assumed to have a unit weight, γ of 16.5 kN/m^3 and coefficient of earth pressure at rest, K_o' of 0.60.

This paper examines results obtained using a construction sequence in which a 375 mm thick granular working mat is first placed directly on the surface of the clay deposit and the geosynthetic reinforcement is then placed on top of this granular working mat. The embankment is subsequently "constructed" to failure and ultimate collapse by placing horizontal lifts of granular fill on top of the geosynthetic. The fill-geosynthetic interface friction angle was taken to be 32°. The finite element model allows for slip at the clay-fill interface below the reinforced embankments as well as slip at the fill-geosynthetic interface.

To determine the embankment fill thickness and geosynthetic strain at failure, factored strength parameters were used in the finite element analyses. For a given foundation with nominal strength parameters c_{uo} and ρ_c, adopting a factor of safety FS=1.3, the factored strength parameters used in the analyses are $c_{uo}^*=c_{uo}/1.3$ and $\rho_c^*=\rho_c/1.3$. Thus, the fill thickness at failure computed using factored parameters corresponds to the allowable fill thickness under working conditions for the nominal strength parameters and a "factor of safety" equal to 1.3.

SOFT BRITTLE CLAY DEPOSITS

Inevitably, two important factors governing the behaviour of embankments constructed on soft brittle clay deposits are the strength characteristics of the underlying foundation and the modulus (stiffness) of the geosynthetic reinforcement. Figure 2 shows the relationship between applied pressure, γH (due to the embankment load, where H is the maximum thickness of fill at a given point in time), and the maximum strain developed in the geosynthetic reinforcement obtained from several finite element analyses.

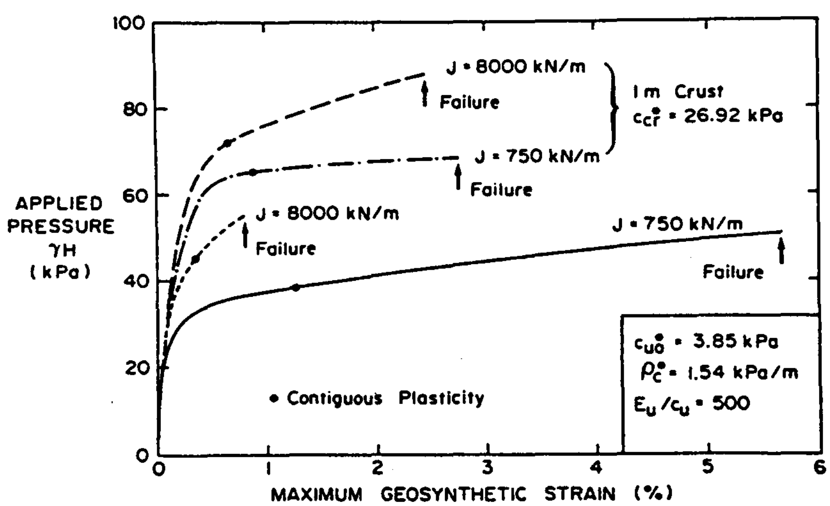

Figure 2. Effect of geosynthetic modulus on the maximum strain in the reinforcement

Analyses were performed for foundations assuming an undrained strength profile which increases with depth at a rate, ρ_c, from some surface strength, c_{uo}. Analyses were performed with and without a higher strength surface crust of strength c_{cr} and thickness 1 m (see also Figure 1 for a definition of the various strength terms). The nominal strength parameters considered were c_{uo}=5 kPa, ρ_c=2 kPa/m and for the profiles with a crust, c_{cr}=35 kPa. Factored parameters c_{uo}*, ρ_c* and c_{cr}* were used in the analyses.

The curves shown in Figure 2 both start at an applied pressure equal to 7.5 kPa because of the assumed construction technique (i.e. placement of 375 mm working mat prior to placement of the geosynthetic reinforcement) and terminate at a maximum geosynthetic strain corresponding to that at failure of the embankment. From a practical standpoint, a reinforced embankment is assumed to have failed at a fill thickness where the increment in vertical displacement is equal to or exceeds the increment in fill thickness just added. The addition of more fill will not result in a net increase in embankment height (for a detailed discussion regarding the failure and collapse of geosynthetic reinforced embankments, refer to Rowe and Soderman (1987)).

The shape of the curves shown in Figure 2 can be explained in terms of the behaviour of reinforced embankments on soft foundations. Initially, the maximum strain in the geosynthetic reinforcement increases slowly with increasing applied pressure and the foundation carries most of the load due to the embankment. However, once contiguous plastic failure occurs in the underlying foundation soil, the reinforcement must carry any additional load which must be resisted along the potential failure surface and hence the geosynthetic strains increase rapidly. The exception to this is the curve shown in Figure 2 for a very stiff reinforcement, J=8000 kN/m, on a very soft foundation with no surface crust (to be discussed in subsequent paragraphs).

To examine the effect of reinforcement modulus on the behaviour of embankments constructed on very soft brittle clay deposits without a surface crust, consider the lower two curves shown in Figure 2. The underlying clay foundation is assumed to have nominal strength properties c_{uo}=5 kPa and ρ_c=2 kPa/m, however factored parameters are used in the analysis (i.e. c_{uo}*=5/1.3 = 3.85 kPa and ρ_c*=2/1.3 = 1.54 kPa/m, where 1.3 is the assumed design factor of safety). The fill thickness (applied pressure) at failure obtained using factored parameters is therefore the allowable design height under working conditions (provided that the permissible strain for the geosynthetic is not exceeded). At failure, the maximum strain is approximately 5.7% for the embankment reinforced with a geosynthetic of modulus, J=750 kN/m and the fill thickness at failure is 2.5 m. This represents a 47% improvement in failure thickness when compared to the unreinforced failure height of 1.70 m. Increasing the modulus to a very high value of say, J=8000 kN/m has resulted in a marginal increase in fill thickness to 2.80 m (i.e. 0.3 m increase) and resulted in a substantially lower strain of 0.8%. The very high modulus reinforcement has significantly altered the behaviour of the embankment system. This is best illustrated by examining Figures 3 to 8 which compare the response of the two reinforced embankments under discussion.

Shown in Figure 3 is the velocity field at failure for the embankment reinforced with a geosynthetic of modulus, J=750 kN/m. The arrows indicate the direction and relative magnitude of soil movement at failure. The shape of the failure mechanism resembles that of a classical slip circle.

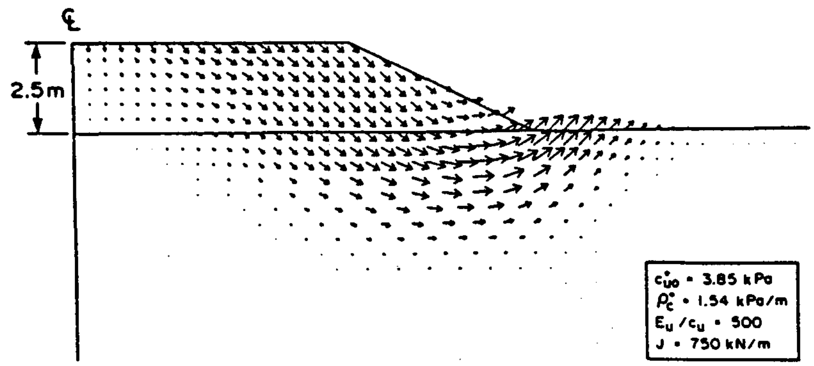

Figure 3. Velocity field at failure; J=750 kN/m

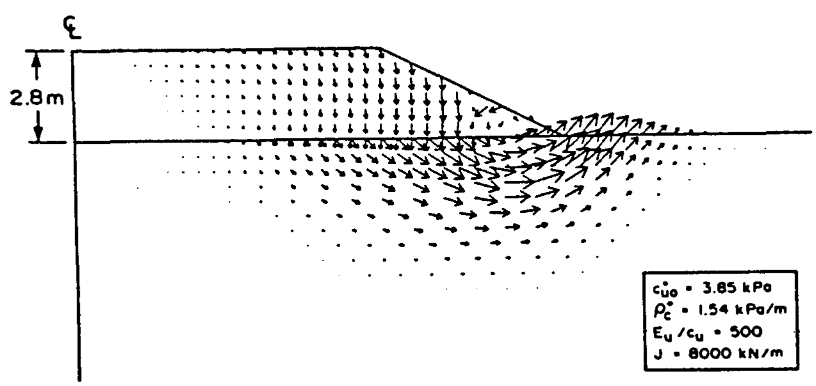

Figure 4. Velocity field at failure; J=8000 kN/m

Increasing the modulus of the geosynthetic to a very high value, J=8000 kN/m results in a velocity field as shown in Figure 4. What is interesting to note is that the inclusion of a very stiff reinforcement has substantially reduced the lateral component of displacement in the embankment fill, which is tending to move downward as a semi-rigid block and there is considerable slip at the fill-clay interface as indicated by the dual arrows shown in Figure 4. The movement of the fill is constrained by the reinforcement and the clay is squeezed out underneath. This type of behaviour is similar to that observed for steel strip reinforced embankments discussed by Rowe and Mylleville (1988) and Mylleville and Rowe (1988). In this case, failure is a bearing capacity type failure (rather than failure based on consideration of deformations). The fill thickness to cause collapse can be estimated reasonably well using a technique proposed by Rowe and Soderman (1987) where the heavily reinforced embankment is idealized as an equivalent rigid footing. Interestingly enough, (based on an examination of the velocity field, Fig.4) it would appear that there is an effective crest width, B^* (which is less than B), corresponding to the failure mechanism. Using this reduced crest width B^*=9 m, the theoretical bearing capacity fill thickness was estimated to be 2.7 m. This compares reasonably well with the fill thickness of 2.8 m at failure for this heavily reinforced embankment which is tending to behave as a semi-rigid footing. It is noted that in this case the more than ten fold increase in reinforcement modulus (from 750 kN/m to 8000 kN/m) has not resulted in a significant increase in failure height because the embankment with a 750 kN/m reinforcement was already relatively close to the maximum capacity imposed by bearing capacity even though with a J=750 kN/m geosynthetic, failure is still a rotational type failure.

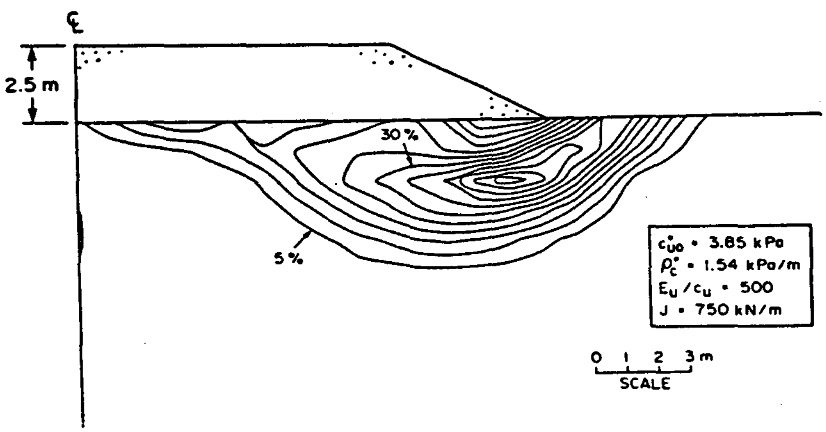

Figure 5. Contours of maximum shear strain at failure: 2.5 m fill thickness - 5% contour interval (from Rowe and Mylleville (1990))

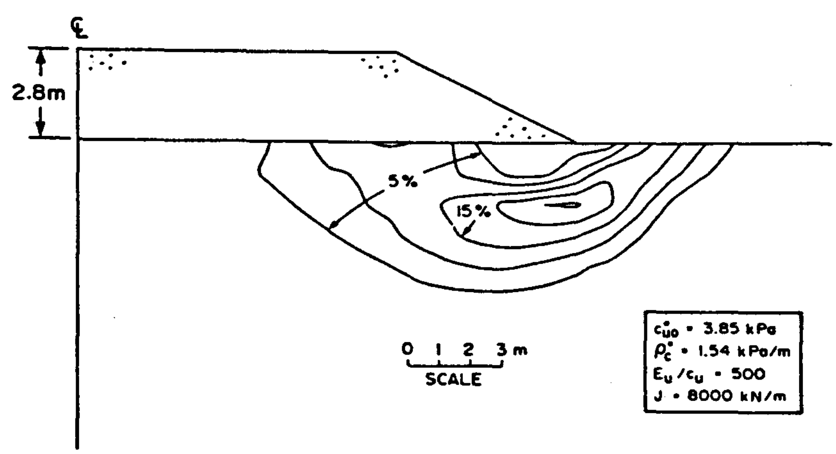

Figure 6. Contours of maximum shear strain at failure: 2.8 m fill thickness - 5% contour interval

Figure 5 shows contours of maximum shear strain at failure in a portion of the underlying clay foundation beneath the embankment reinforced with a geosynthetic having a modulus, J=750 kN/m. The contours for J=8000 kN/m are shown in Figure 6. For purposes of further discussion, maximum shear strain, ϵ_{sh}, is defined as the maximum engineering shear strain in the soil or in other words, the diameter of the Mohr's circle of strain (i.e. $\epsilon_{sh}=\epsilon_1-\epsilon_3$) expressed in percent. The first thing to note is that in both cases, at failure a band of intense shearing develops which is coincident with the failure mechanism in the underlying foundation soil (see also Rowe and Mylleville (1989)). At failure, for the embankment reinforced with a geosynthetic having a modulus, J=750 kN/m, there is a band of foundation soil which experiences maximum shear strains in excess of 30%. At that point, the maximum strain in the geosynthetic was found to be 5.7%. On the other hand, for the case of a very stiff reinforcement, say J=8000 kN/m, a significant band of foundation experiences maximum shear strains in excess of 15% as shown in Figure 6. The maximum strain in the geosynthetic was found to be 0.8%. Thus the stiffer reinforcement has slightly increased the fill thickness to cause failure but at the same time it has significantly reduced the shear strains experienced by the foundation soil.

Various investigators (La Rochelle et al. (1988); Lo and Morin (1972)) have found that sensitive clays may reach peak strengths at axial strains of 1% or less. Under undrained (constant volume) conditions this would correspond to a maximum shear strain, ϵ_{sh} approximately equal to 1.5%. Development of significant zones of shear strain in excess of 1.5% may pose potential problems with respect to strain softening and the results from conventional finite element or limit equilibrium analyses may be unconservative. It becomes evident from Figures 5 and 6 that the failure heights obtained assuming a plastic soil would overestimate the true failure height for a brittle soil since extensive softening would be expected to have occurred. Problems with softening may also arise under working conditions.

Shown in Figures 7 and 8 are the contours of maximum shear strain under working conditions (i.e. analyses were performed using nominal (unfactored) parameters) for the two cases under consideration. As can be seen from Figure 7, for an allowable fill thickness based on a limiting strain in the order of 5% and a geosynthetic of modulus, J=750 kN/m, there is an extensive zone of foundation soil which experiences maximum shear strains in excess of 1.5%. This zone is large enough to create problems if the soil is susceptible to softening at 1.5% strain. The calculated maximum strain in the geosynthetic for the "allowable fill thickness" under working conditions is only 1.3%. It should be noted that these "working conditions" are based on a stability calculation that considers peak strength, neglect strain softening and permits a strain in the geosynthetic at failure of 5 to 6%. As previously discussed by Rowe and Mylleville (1990), adopting a lower limiting strain of say 2% to obtain an allowable fill thickness from a conventional plastic analysis may help to reduce the maximum shear strain developed in the foundation soil to a more acceptable level.

Plotted in Figure 8 are contours of maximum shear strain under working conditions for the very high modulus reinforcement, J=8000 kN/m. The inclusion of a very stiff geosynthetic has dramatically reduced the extent of the zone of foundation soil which experiences shear strains in excess of 1.5%. The maximum strain in the geosynthetic under working conditions is only 0.3%. If the embankment height had been limited to 2.5 m (which corresponds to 0.5% maximum strain in the geosynthetic using factored strength parameters; see Fig.2) then under working conditions (i.e. using nominal strengths) the maximum calculated shear strains are only about 1%.

A general observation which can be made is that for the soft brittle clay deposit just discussed, a very high modulus geosynthetic (e.g. J=8000 kN/m) may not be required to realize a substantial improvement in failure height compared to the unreinforced case. However, keeping in mind that the very high modulus reinforcement may change the failure mechanism and may warrant special consideration from a design standpoint, it does however significantly reduce the magnitude of maximum shear strains experienced by the foundation soil.

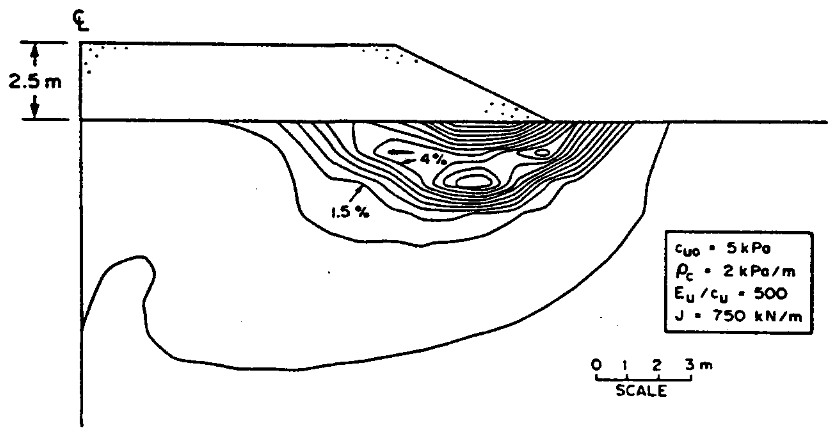

Figure 7. Contours of maximum shear strain under working conditions: 2.5 m fill thickness - 0.5% contour interval (from Rowe and Mylleville (1990))

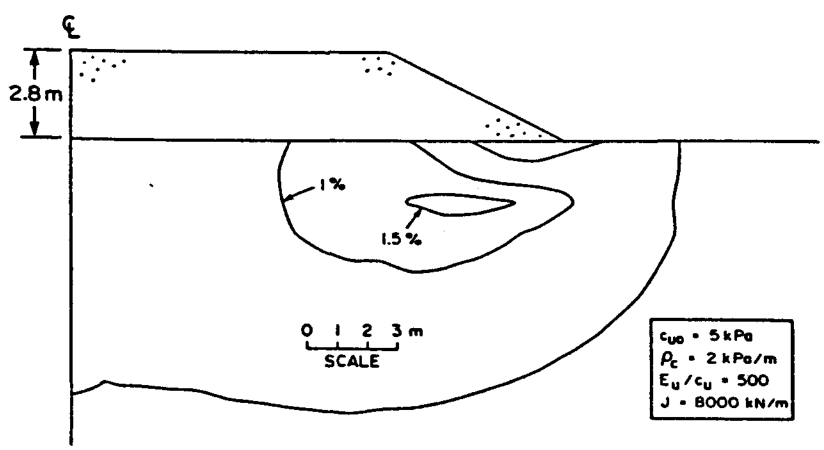

Figure 8. Contours of maximum shear strain under working conditions: 2.8 m fill thickness - 0.5% contour interval

SOFT BRITTLE CLAY DEPOSITS WITH A SURFACE CRUST

Many of the soft brittle clay deposits encountered, for example in eastern Canada and Scandinavia have a higher strength surface crust. Humphrey and Holtz (1989) and Rowe and Mylleville (1990) have demonstrated that a surface crust can substantially improve the stability of embankments. The presence of a higher strength surface crust may have a number of beneficial effects when compared to the case without a crust. The higher strength surface crust may help to increase the allowable embankment fill thickness, reduce the strain in the reinforcement and reduce the magnitude of maximum shear strains experienced by the foundation soil.

In this section, results of finite element analyses are presented which examine the effect of geosynthetic modulus on embankment performance. Two geosynthetic modulus values are examined: J=750 kN/m and J=8000 kN/m. The clay foundation is assumed to have a 1 m thick surface crust with a nominal undrained shear strength, c_{cr}=35 kPa. Beneath the crust the strength profile increases with depth at a rate of 2 kPa/m from an initial value of 7 kPa directly under the 1 m crust. The analyses were performed using factored parameters therefore the fill thickness at failure corresponds to the allowable fill thickness under working conditions.

One should keep in mind that for analyses with a high stiffness/strength surface crust, tensile stresses are attracted to the crust. Close examination of the results did show some tensile stresses to exist outside of the embankment, however only in the extreme upper portion of the surface crust. If the tensile stresses had propagated through the crust into the underlying soil, then this would constitute failure. Nevertheless, the results presented here should be regarded as upper bound solutions. If the crust was naturally fractured to any significant extent, then the improvement discussed below may not be fully realized.

The upper two curves in Figure 2, show the relationship between applied pressure and maximum geosynthetic strain for the analyses which will be discussed. The embankment reinforced with a geosynthetic of modulus J=750 kN/m, failed at a fill thickness of 3.4 m. The fill thickness at failure for the unreinforced case was found to be 3.2 m , hence it would seem the influence of the crust dominates with only a minimal improvement in failure load due to the inclusion of the geosynthetic.

The use of a higher modulus reinforcement, say J=8000 kN/m resulted in a fill thickness of 4.4 m at failure which represents a 38% improvement in failure load compared to the unreinforced case. However, one observation which can be made looking at Figure 2, is that the high modulus reinforcement (i.e. J=8000 kN/m) results in a somewhat more ductile response (i.e. beyond contiguous plasticity, the geosynthetic strain increases at a slower rate with increasing load to failure).

Figure 9 shows contours of maximum shear strain at failure for the case of a geosynthetic with modulus J=750 kN/m. Within the zone of intense shearing, the soil experiences maximum shear strains in excess of 10%. The contours of maximum shear strain at failure for the case of the very stiff geosynthetic are shown in Figure 10. The maximum shear strains experienced by the foundation soil are in excess of 14%, however the failure load is greater compared to that of the previous case (i.e. 4.4 m viz. 3.4 m). It should be noted that the maximum shear strains within the zone of intense shearing are comparable in magnitude to those shown in Figure 6 for the analysis without a crust and a considerably lower failure load. This once again illustrates the beneficial effect of a surface crust.

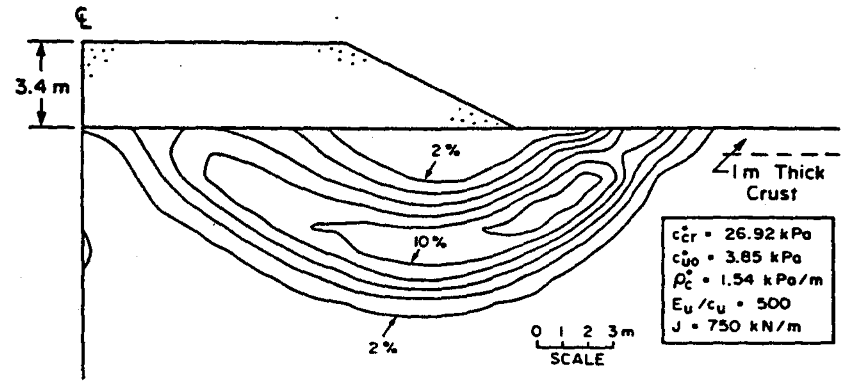

Figure 9. Contours of maximum shear strain at failure: 3.4 m fill thickness - 2% contour interval (from Rowe and Mylleville (1990))

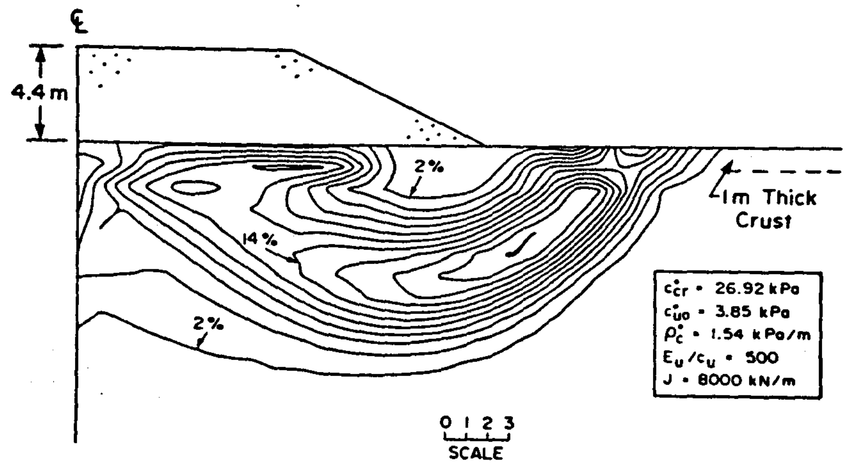

Figure 10. Contours of maximum shear strain at failure: 4.4 m fill thickness - 2% contour interval

Contours of maximum shear strain under working conditions for J=750 kN/m and J=8000 kN/m are plotted in Figures 11 and 12 respectively. For the case of J=750 kN/m, the maximum value is just in excess of 1%, whereas for the case of J=8000 kN/m the value is in excess of 1.6% (keeping in mind that in the latter case, the foundation soil is subjected to 38% more load). Looking at Figure 12, there is a significant zone of soil where the calculated shear strains are in excess of 1.5%, hence the potential for problems exists if the foundation soil is susceptible to strain softening. Adopting a lower limiting strain of say 0.5% in design (see Fig. 2), the calculated shear strains which correspond to a fill thickness of 3.5 m are plotted in Figure 13. The maximum value is just in excess of 1%. This suggests that adopting a 2% limiting strain in design for the very stiff geosynthetic may be unconservative, since the foundation soil may be subjected to shearing strains large enough to cause problems with softening.

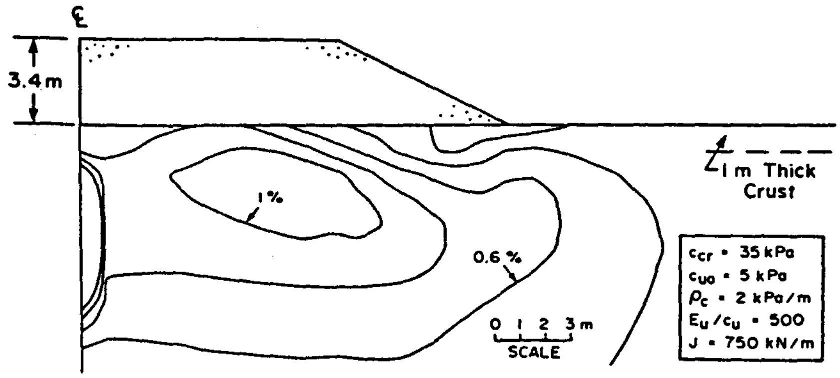

Figure 11. Contours of maximum shear strain under working conditions: 3.4 m fill thickness - 0.2% contour interval (from Rowe and Mylleville (1990))

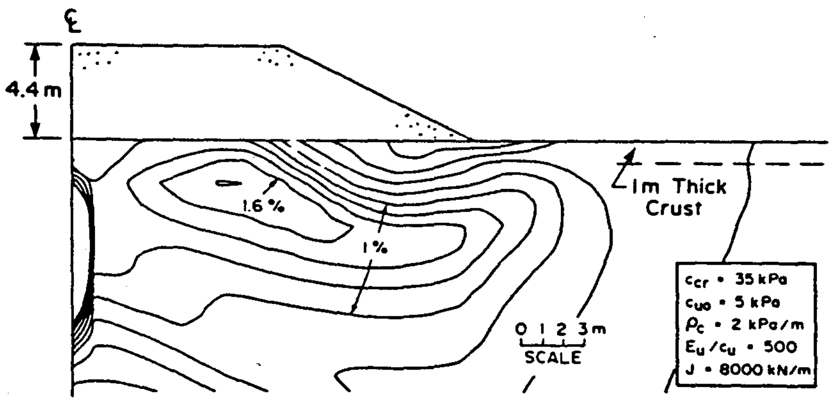

Figure 12. Contours of maximum shear strain under working conditions: 4.4 m fill thickness - 0.2% contour interval

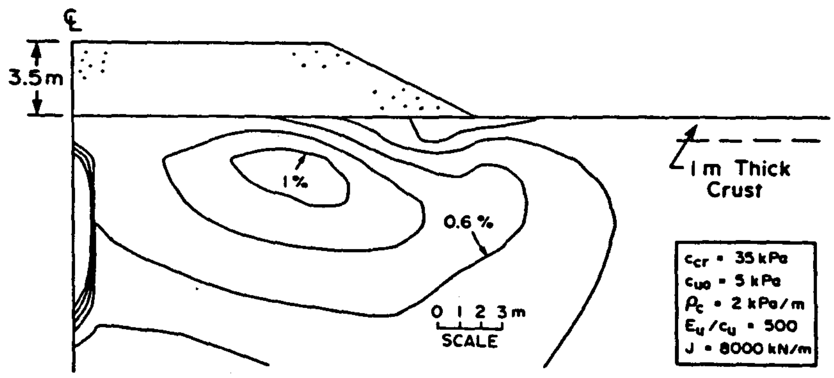

Figure 13. Contours of maximum shear strain under working conditions: 3.5 m fill thickness - 0.2% contour interval

IMPLICATIONS FOR PRACTICAL DESIGN

A number of results of finite element analyses have been presented which illustrate some important points one should keep in mind when designing reinforced embankments on soft brittle clay deposits. Calculated values of maximum shear strain in the foundation soil have been examined to assess the potential for strain softening.

Based on the results discussed, it would seem that for very soft brittle soils without a crust that the use of a very high modulus (i.e. J=8000 kN/m compared to J=750 kN/m) results in a greater fill thickness and reduction in shear strains in the foundation soil under working conditions. This may not be particularly significant for plastic soils but may be important if the soil is susceptible to softening. The use of a very high modulus reinforcement tends to change the failure mechanism to a bearing capacity type problem with significant slip of the foundation soil beneath the heavily reinforced embankment. As a result, the strain mobilized in the reinforcement tends to be quite low. Adopting a limiting strain in the order of 2% as recommended in the literature for sensitive brittle soils (Bonaparte and Christopher (1987)) may lead to unconservative designs in some cases. A limiting strain of say 0.5% may be more realistic for high modulus geosynthetics on soft soils susceptible to strain softening if one is going to design based on peak strength.

For soft brittle soils with a surface crust, it would seem that very little improvement in fill thickness is realized by the inclusion of a geosynthetic with a modulus, J=750 kN/m. The presence of the higher strength crust seems to dominate. On the other hand, the use of a very stiff geosynthetic, J=8000 kN/m results in a significant improvement in allowable fill thickness. However, under working conditions, the improvement in fill thickness results in calculated shear strains which may be large enough to cause problems if the foundation soil is susceptible to softening. Again, for the very high modulus geosynthetic, a limiting strain as low as 0.5% may be necessary to reduce the calculated shear strains to an "acceptable" level. This low value of limiting strain results in an allowable fill thickness which is for all practical purposes the same as that for the lower modulus geosynthetic.

It should be emphasized that the foregoing analyses and discussion assumed that the design was based on peak strength. An alternative would be to use post-peak strengths corresponding to the expected strains or in this case the limit on allowable strain would be higher and the strength would be lower. The designer could assess the economic implications of either design to "prevent softening" or design to accept softening and use post-peak strength.

CONCLUSIONS

The results of finite element analyses have been used to examine the effect of geosynthetic modulus on the behaviour of reinforced embankments constructed on very soft brittle clay deposits. A rigorous examination of strains in the geosynthetic and underlying foundation soil has resulted in a number of interesting observations.

It was found that increasing the geosynthetic modulus from J=750 kN/m to J=8000 kN/m resulted in an improvement in failure load and a significant reduction in shear strains in the underlying foundation soil. However, for the high modulus geosynthetic (i.e. J=8000 kN/m), the reinforcement strains at failure are very low for a very soft deposit without a surface crust. The failure mechanism for the heavily reinforced embankment resembled that of a semi-rigid footing bearing capacity failure.

If the foundation soil is susceptible to strain softening then care must be taken in a design situation. Limiting reinforcement strains recommended in the literature may lead to designs which subject the soil to excessive shear strains, even under working conditions. Results are presented which suggest the use of a limiting strain as low as 0.5% in a conventional plastic analysis may be warranted if a very stiff reinforcement is used.

It has been demonstrated that for soft brittle soils with a higher strength surface crust, the effect of the crust dominates, even if a very high modulus geosynthetic is used. For the cases considered, it was found that under working conditions, the modulus of the geosynthetic had very little effect on the magnitude of calculated shear strains in the foundation soil. In other words, keeping the shear strains low to avoid problems with strain softening, resulted in a failure load (fill thickness) which was for all practical purposes the same whether a geosynthetic with a modulus of J=750 kN/m or J=8000 kN/m was used.

ACKNOWLEDGEMENT

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