Second International Conference of Geotextiles, Las Vegas, U.S.A.

ROWE, R. K.
University of Western Ontario, London, Canada

The Analysis of an Embankment Constructed on a Geotextile

Etude d'un remblai construit sur géotextile

A technique for the analysis of geotextile reinforced embankments is used to examine the factors affecting the performance of a test embankment constructed at Pinto Pass, Alabama. The analysis includes consideration of the development of membrane forces, variable foundation support characteristics, internal embankment arching and load redistribution, plastic failure of the foundation, slip at the soil-fabric interface and simulation of complex construction sequences.

The paper studies the effects of construction sequence, fabric stiffness, underlying soil properties and fill stiffness upon the embankment performance. Particular consideration is given to the effects of the geotextile upon settlements, horizontal movements, membrane forces and embankment stability. The implications of the theoretical predictions and the observed behaviour are briefly discussed.

Nous utilisons une méthode pour analyser les remlais renforcés de fibre geotextile pour étudier les facteurs qui ont pu affecter la performance d'un essai entrepris à Pinto Pass, Alabama. L'analyse considère le développement des forces de membrane, des caractéristiques de support de fondation variables, la redistribution interne de poids, la rupture de fondation en régime plastique, le glissement à l'interface sol-fibre et al simulation de séquences complexes de construction.

Nous étudions dans cette publication les effets de la séquence de construction, de la rigidité du fibre et des propriétés du sol sur la performance d'un remblai. Nous discutons brièvement les implications des résultats théoriques ainsi que le comportement observé.

INTRODUCTION

Geotextiles are currently finding acceptance as a means of facilitating the construction of embankments on soft/weak foundations. Much of the literature relating to the effect of geotextiles upon the performance of these embankments is conceptual in nature and, as yet, has not been fully supported by experimental and theoretical studies. Design methods based on these concepts (eg. (5) and others) involve relatively straightforward extensions of basic engineering concepts relating to earth pressure and slope stability. From an engineering standpoint, these methods are very attractive as they require relatively little input data and the calculations required for fabric selection are extremely simple and straightforward. However, it must be admitted that these approaches do make a number of arbitrary assumptions and it is of some scientific and practical interest to examine the mechanisms affecting the behaviour of these embankments, using more sophisticated analytical techniques.

The Fabric Reinforced Test section constructed at Pinto Pass, Mobile Alabama (4), (7) has played a major role in the development of data and concepts for uses in the design of geotextile reinforced embankments. Consequently in this paper, a finite element soil-structure interaction analysis will be used to examine the behaviour of this test section so as to provide some additional insight into the role of the geotextile.

PRINCIPAL ASSUMPTIONS AND METHOD OF ANALYSIS

The results presented in this paper were obtained using the plane strain, Elasto-Plastic Soil-Structure Interaction Analysis program EPSSIA which is based on the general soil-structure interaction technique proposed by Rowe et al (8). In this analysis, the soil is assumed to be an elastic-plastic material with a Mohr-Coulomb failure criterion (defined in terms of the cohesion c and angle friction ϕ) and a flow rule of the form proposed by Davis (2). For the analyses reported herein, it was assumed that the soil deformed plastically at constant volume, however, provision is made for plastic dilation of the soil if this is considered appropriate. The geotextile was treated as a structural membrane with axial stiffness but negligible flexural rigidity. Provision was made for slip between the fabric and the soil above and/or below the fabric. The displacement of the soil and fabric were assumed to be compatible until the shear stress reached a limiting shear stress defined by a Mohr-Coulomb criterion at the interface. Once this shear stress was attained, slip (ie. differential tangential displacement between the soil and fabric) occurred at this point.

Since the geotextile is considered to develop membrane forces due to deformation, the co-ordinates of the fabric were updated during the analysis. To be consistent, a large deformation analysis was also performed for the soil.

It is noted that the use of this soil-structure interaction approach allows consideration of: the development of membrane forces; variable foundation support characteristics (including local failure); permits the development of internal embankment arching and load redistribution caused by soil displacement;

slippage at the soil-fabric interface if, and only if, the interface shear strength is reached; and permits simulation of complex construction sequences including the placement of the fabric and lapping back of the edge of the fabric to provide anchorage (ie. the approach does not require any arbitrary assumption regarding the anchorage of the fabric). This approach allows the determination of embankment deformations, membrane forces and stability.

PINTO PASS CASE

The design, construction and performance of the Pinto Pass test section has been described in several publications (4), (6) and (7). Briefly, a 2.4 m high, 253 m long and 52.4 m wide test embankment was constructed with a 3.6 m wide crest and 10:1 (horizontal: vertical) side slopes on a foundation of very soft, highly plastic clays and loose clayey fine sands and silts which extended to depths of from 9.75 m to 12 m, where dense clean sand was encountered. In this paper attention will be restricted to a "typical" cross-section of the embankment which, on the basis of limited soils data, may be considered to be underlain by approximately 3.6 m of soft clay (CH), 4 m of clayey and silty sand (SM-SC) and a further 2.15 m of "fairly strong highly plastic clay" (CH) (4) which rested on medium to dense sand. Field vane shear tests indicated undrained shear strengths of approximately 2.4 kPa to a depth of 1.5 m, 4.8 kPa from 1.5 to 2.4 m and 7.2 kPa from 2.4 m to 3.6 m. The embankment was successfully constructed in four sections using four fabrics with properties as indicated in Table 1. The observed settlements of this embankment was typically 0.3 m and in the worst case 0.5 m.

Table 1. Properties of Geotextiles (after Haliburton et al (6). From uniaxial tests using 152 mm wide x 305 mm long samples.

Fabric	Ultimate (kN/m)	Strain at Failure%	Initial Tangent Modulus (kN/m)	Secant Modulus 10%ε(kN/m)
Nicolon 66475	158.0	21	125	634
Polyfiller X	54.5	35	250	180
Advance Type I	44.0	29	613	188
Nicolon 66186	39.6	15	46	190

SELECTION OF PARAMETERS FOR USE IN THE ANALYSIS

The published data relating to the Pinto Pass case involves considerable uncertainty with regard to the soil and fabric properties. Thus to provide some insight into the performance of this embankment, a limited parametric study was performed involving four different representations of the soil profile as indicated in Table 2, and five different values of fabric modulus. It was assumed that: the soil had a unit weight of 14.5 kN/m^3 and K_o = 0.6; the fabric soil interface had a friction angle of 30°; and the soil profile consisted of horizontal layers (so that symmetry could be assumed). Although data supporting this latter assumption is scarce, the approximately symmetric settlements observed beneath most of the embankment would indicate that the assumption is reasonable except at sections 5+00 and 6+00 where the settlements were markedly non-symmetric (this is probably due to the oblique intersection of an old channel with the embankment).

Soil profiles [1], [2] and [4] adopt undrained cohesions c_u based upon the published data to a depth of 5.5 m; there is no direct data below 5.5 m. In profiles [1] and [2] it is assumed that the entire deposit is uniform and normally consolidated and on this basis the undrained shear strength below 5.5 m may be estimated from the available data. In profile [1] the elastic modulus of the soil is assumed to be 1000 c_u and corresponds to undrained conditions.

Strictly speaking, a full effective stress elasto-plastic consolidation (1) analysis should be used to

Table 2. Soil Profiles Examined. (1) ϕ=0°; (2) ϕ=25°

Profile Depth (m)	[1] E kPa	[1] c kPa (1)	[2] E kPa	[2] c kPa (1)	[3] E kPa	[3] c kPa (2)	[4] E kPa	[4] c kPa	[4] ϕ
0 - 0.6	2400	2.4	120	2.4	120	1.9	350	2.4	0
0.6-1.5	2400	2.4	180	2.4	180	1.9	350	2.4	0
1.5-2.4	4800	4.8	240	4.8	240	1.9	500	3.4	0
2.4-3.6	4800	4.8	300	4.8	300	1.9	700	4.8	0
3.6-5.5	7200	7.2	390	7.2	390	2.9	5500	0.5	35°
5.5-7.6	13200	13.2	500	13.2	500	5.75	7000	0.5	35°
7.6-9.75	16800	16.8	600	16.8	600	5.75	7000	16.8	0

obtain the final settlement since this will model the initial undrained response followed by subsequent dissipation of pore pressures. However, it has been found that in predicting the final settlement of embankments of soft soil, it is generally adequate to avoid the consolidation phase by using undrained shear strength parameters and drained elastic parameters (3) (this inconsistent use of undrained and drained parameters is necessary if the approximate approach is to simulate both the plastic strains that develop under undrained conditions as well as the consolidation that occurs as pore pressures dissipate). Thus profile [2] adopts the same shear strength as profile [1] but the elastic moduli are determined assuming a homogeneous deposit to 9.75 m and calculating an equivalent E for the appropriate stress range assuming a uniform normally consolidated deposit with average properties e_o = 2.7 and C_c = 0.8. (Thus profile [2] approximately corresponds to the profile used by Fowler (4, p. 67) for settlement calculations which suggested a settlement of the embankment, without fabric, of 0.91 m (3 ft.). Profile [3] uses both drained shear strengths and modulus values consistent with the soil profile assumed in profiles [1] and [2].

Profile [4] is considered to be the most realistic case. It assumes soft clay to a depth of 3.6 m underlain by 4 m of clayey sand which is assumed to have cohesive frictional properties. The clayey sand is underlain by another 2.15 m of medium clay. The modulus values adopted for this profile were determined from empirical correlations based on the author's previous experience in predicting embankment behaviour.

Table 1 indicated a wide variation in fabric stiffness depending upon the type of fabric and the strain. In addition, it is noted that these results are from uniaxial tests whereas in practice the fabric will be acting in an approximately plane strain mode. Thus despite this available test data, the actual operational stiffness is unknown. To provide an indication of the effect of fabric stiffness, four fabric moduli were considered. The basic case ("E" = 125 kN/m) corresponds to the magnitude at the lower end of the range reported in Table 1. The second (625 kN/m) represents a 5 fold increase and is at the higher end of reported range. The third and fourth cases correspond to a 10 and 100 fold increase in stiffness over the basic case. As a control, analyses were also performed for a fabric with negligible stiffness.

The fill material was assumed to be non-cohesive with a friction angle of 30° and a unit weight of 15.7 kN/m^3. The Youngs modulus E of the fill was defined by an equation of the form $(E/p_a) = K(\sigma_3'/p_a)^n$ in which σ_3' is the minor principal effective stress; p_a is atmospheric pressure; and K, n are two coefficients appropriate for loose material at low confining stresses. Analyses were performed for a number of values of (K, n) namely [A] (57, 0.51); [B] (450, 1.0); [C] (450, 0.5). The first two envelopes were based on triaxial tests performed by the author and his co-workers on loose samples of different sands at appropriate stress levels; the third envelope is based on data from the literature and the density and stress levels may be less appropriate.

Unless otherwise noted, the following results were obtained for the basic fabric (E = 125 kN/m) and fill properties [B] (K = 450, n = 1).

RESULTS

Soil Parameter Selection and Analysis

The effect of the soil profile upon settlements at points on the soil surface located at the centreline and 13.7 m from the centreline, as well as the mobilization of fabric force are shown in Fig. 1. The

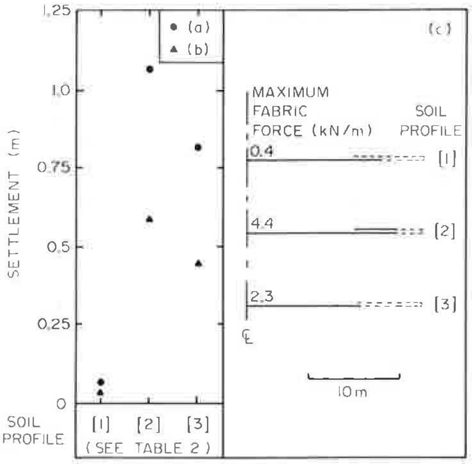

Fig. 1 Variation in Settlement and Fabric Force with Soil Profile (Fabric Stiffness 125 kN/m)
(a) Settlement at Centreline
(b) Settlement 13.7 m from Centreline
(c) Schematic of the Fabric; full line indicates the portion of the fabric where tension is mobilized.

undrained profile [1] gives a relatively small settlement and fabric force. An undrained analysis may be useful in assessing embankment stability, but is unlikely to provide a good indication of the effect of the fabric upon general embankment performance. Because of the much smaller "elastic" modulus of the soil, the fully drained analysis (soil profile [3]) gives much larger settlements and fabric force than the undrained analysis, although the extent of tension in the fabric is slightly less. The drained analysis did not involve any plastic failure within the soil, and hence, all the deformations may be considered to be due to consolidation. This type of calculation provides a lower estimate of the settlement. In practice, local yield will probably occur in the soil mass during construction. Thus the use of the undrained shear strength parameters together with the drained "elastic" parameters (soil profile [2]) provides an indication of undrained stability as well as an upper estimate of the deformations of the embankment (neglecting creep). This analysis does not give the exact final settlement, however, it is considered to be sufficiently accurate (and conservative) for practical purposes; particularly when the uncertainty regarding parameters is considered.

Construction Sequence

Embankment construction was split into 60 steps, and as nearly as possible, followed the construction procedure described by Haliburton et al (7). As construction proceeded, plastic regions developed both within the embankment and within the underlying soil, as illustrated in Fig. 2 for four stages during the construction (soil

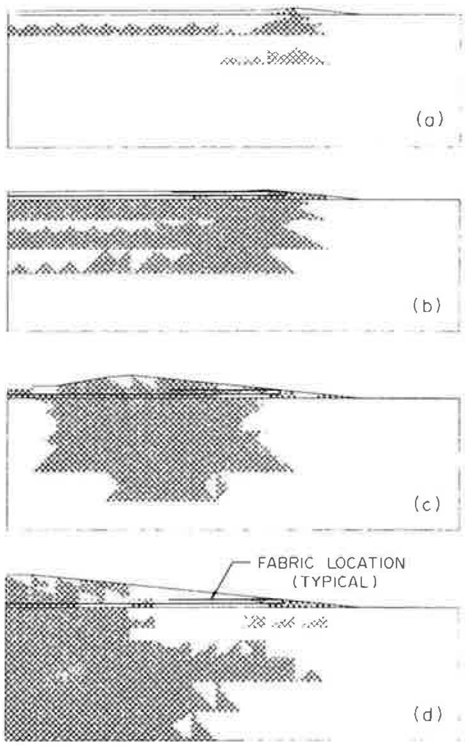

Fig. 2 Plastic Zones at Four Stages During Construction. Soil Profile [2]; Fabric Stiffness 125 kN/m.

profile [2]). Because of the outside-inside construction sequence, some regions of the soil which are plastic at early stages in the construction experience a decrease in deviator stress (and cease to be plastic) at later stages in the construction sequence and this has a beneficial effect upon the general response of the embankment. The plastic region at full construction extends down to the dense sand layer. However, this plasticity is contained and the factor of safety is greater than unity.

A conventional construction sequence involving construction of the embankment in horizontal layers was also considered. No difficulty was encountered in developing the fabric anchorage, although the shear stress was mobilized over a shorter distance. No shear slip at the fabric-soil interface occurred for either construction sequence. However, the construction sequence proposed by Haliburton et al (7) was superior as it gave rise to settlements 10% smaller than obtained using the conventional approach.

Effect of Fabric Stiffness

The vertical settlements at the natural soil surface for points on the centreline and 13.7 m from the centreline are shown in Fig. 3 for a range of fabric stiffnesses

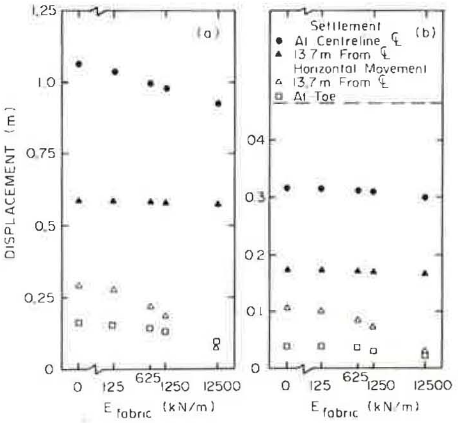

Fig. 3 Variation in Displacement as a Function of Fabric Stiffness for Two Soil Profiles (a) Soil Profile [2], (b) Soil Profile [4].

and two soil profiles. Also shown are the lateral displacements of the 13.7 point and at the toe of the embankment (note that the total lateral spreading will be twice the value shown due to symmetry). Profile [2] gives a centre settlement of 1.06 m for the no fabric case. This is slightly larger but of a similar order to that calculated by Fowler (4) using simple settlement theory and assuming a similar soil profile. However, an inspection of Fig. 3a shows that increasing the fabric stiffness has a very modest effect upon settlements and even the use of an extremely (perhaps unrealistically) stiff fabric only reduces the centreline settlement by 10% to 0.93 m. The differential settlement was also reduced by increasing fabric stiffness but again for this combination of soil and fill moduli, the effect of the fabric was not large. The major effect of the fabric stiffness was upon the horizontal displacements. For the typical range in reported fabric stiffness the horizontal movement was reduced by between 7% and 25%, and by upto 75% over the entire range of stiffness considered. It is of some interest to note that the displacements vary in an approximately linear fashion with the logarithm of the fabric stiffness.

Similar trends were obtained with soil profile [4].

In this case, the settlement of 0.31 m determined without fabric is very close to the observed settlements at Pinto Pass. However, the settlement was reduced by less than 6% for even the stiffest fabric considered. Again the effect of fabric stiffness was greatest for lateral movements which were reduced by between 7% and 21% for the reported range of fabric moduli, and over 70% for the stiffest fabric.

The mobilization of tension within the fabric, and the maximum fabric force are shown in Fig. 4. The soil

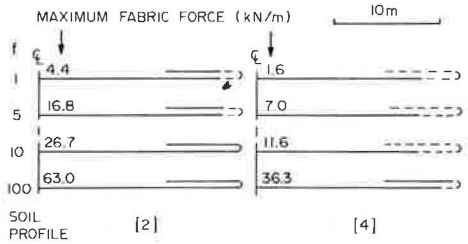

Fig. 4 Development of Tension in Fabric as a Function of Soil Profile and Fabric Stiffness. Tensile forces mobilized in full line region Fabric stiffness = 125f kN/m.

stiffness influences both the degree of tensile mobilization and the magnitude of the maximum tension, however, the results for both the soil profiles suggest that failure in the fabric is unlikely and that the outside edges of the fabric are not stressed (both these findings accord with the observed behaviour). For the typical range of reported fabric stiffness (E = 125 - 625 kN/m) and ultimate tensile capacity (40 - 160 kN/m) the factor of safety against fabric failure ranges between 2.4 and 36 for soil [2] and, more probably, between 5.7 and 100 for soil [4]. The force in the fabric and the degree of mobilization increased with increasing fabric stiffness. Only for the unrealistic stiffness E = 12500 kN/m was the fabric fully tensioned and then only for the large deformations associated with profile [2]. A number of analyses were performed without any fabric overlap and as might be expected, these results were not significantly different from those reported above for the practical range of fabric stiffness.

The foregoing theoretical results would suggest that sufficient fabric anchorage could be mobilized without overlapping the fabric at the edge of the embankment. A similar conclusion was reached by Haliburton et al (7) from consideration of the field performance of the embankment.

The effect of the fabric stiffness upon the development of local yield within the soil is illustrated in Fig. 5 for soil [4]. A comparison of Figs. 5a and 5b indicates that the presence of the basic fabric (E = 125 kN/m) has no significant effect upon the plastic region. In both cases, the embankment would appear to be marginally stable. With the absence of the fabric, the embankment has a factor of safety of approximately 1.13. This factor of safety was determined by reducing the underlying soil shear strength until failure of the embankment occurred and may be compared with the factor of safety of 1.07 obtained from a modified Bishop analysis. This analysis included the shear strength of the embankment (the factor of safety is less than unity without the fill strength). In design it is common, and conservative to neglect the fill strength and stiffness to allow for the possibility of tension cracks, however no tension cracking was reported and the results of this

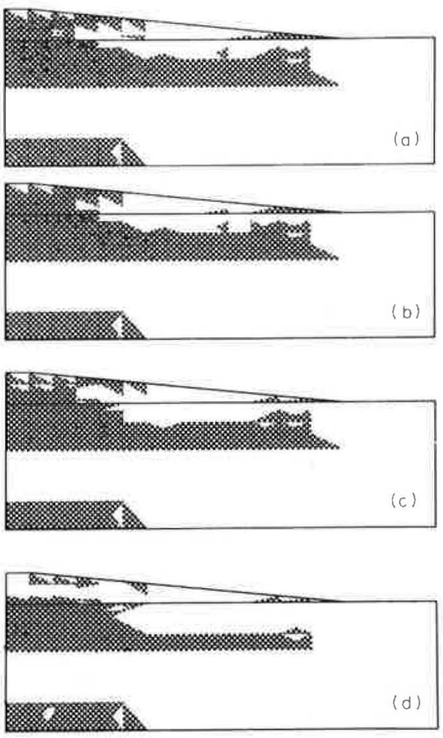

Fig. 5 Plastic Regions After Full Construction for Four Fabric Stiffnesses (a) No fabric (b) E = 125 kN/m (c) E = 1250 kN/m (d) E = 12500 kN/m. Soil Profile [4].

analysis did not indicate the development of tensile failure within the embankment. Hence, for the purposes of interpreting the behaviour of this embankment, it is considered appropriate to include the fill strength in the analysis.

Increasing fabric stiffness above the basic value tends to reduce the extent of local yield although the effect is not particularly evident until the fabric stiffness is 1250 kN/m. (Fabric stiffness has a somewhat greater effect upon the plastic regions for soil profile [2]). The effect of the fabric upon embankment stability may be assessed by reducing the underlying soil strength until uncontained plastic flow occurs. Fig. 6 shows the plastic regions corresponding to a decrease in soil strength such that if failure just occurs, it would imply a factor of safety of 1.25.

Fig. 6 indicates that with a fabric stiffness of 625 kN/m, there is uncontained plasticity and an inspection of the associated velocity field (see Fig. 7) indicated impending failure. The lateral spreading of the embankment has increased more than two fold, however, the tensile capacity has not yet been reached and in that sense, failure has not occurred. Thus a slip circle analysis which incorporates a restoring moment due to the tensile capacity of the fabric would not indicate failure. Nevertheless, the embankment is clearly in severe distress as soil is squeezed out from between the embankment and the stronger underlying clayey sand. Due to the relatively low stiffness of this fabric, extremely large

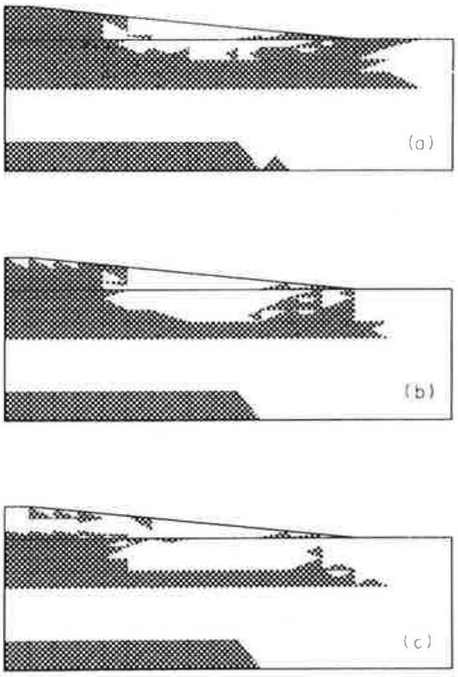

Fig. 6 Plastic Regions at Full Construction for Three Fabric Stiffness (a) E=625 kN/m (b) E=1250 kN/m (c) E=12500 kN/m. Soil Profile [4] except that all shear strengths have been divided by 1.25 [failure at full construction for this case implies a Factor of Safety of 1.25].

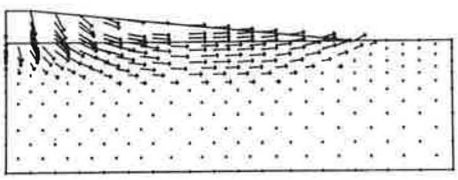

Fig. 7 Velocity Field Corresponding to the Plastic Region Shown in Fig. 6a. Impending Failure.

deformations will occur prior to the fabric reaching its tensile capacity and hence for all practical purposes, failure may be deemed to have occurred prior to rupture of the fabric.

The preceding discussion implies that the stiffness of the fabric may be as important as the tensile capacity in assessing the stabilizing influence of the fabric. This is illustrated by comparing the plastic region in Fig. 6 obtained for three fabric stiffnesses. The major role of the fabric is the reduction of lateral spreading, the stiffer the fabric the less the lateral spreading and the greater the confinement of the underlying soil which in turn reduces plasticity in this soil. If failure is considered to have occurred with the onset of uncontained plastic flow (even though rupture of the fabric may not have occurred) then the factor of safety corresponds to the stiffnesses 125, 625, 1250 and 12500 kN/m respectively are 1.18, 1.25, 1.44 and more than 2.25. Failure of the underlying foundation prior to failure of the fabric may be considered to be a ductile failure, while collapse corresponding to failure of the fabric may be expected to be relatively brittle. Thus a knowledge of both the fabric stiffness and tensile capacity determined under plane strain conditions is considered to be very important for the rational design of geotextile reinforced embankments. With this data, it should be possible to design embankments with a primary factor of safety against ductile failure (ie. failure of the foundation) and with a somewhat higher factor of safety against brittle failure (ie. snap of the fabric).

Effect of Fill Stiffness

To illustrate the effect of fill stiffness, analyses were performed for three sets of the parameters (K, n) which define fill modulus as a function of minor principal stress. The results from these analyses indicated that increasing fill stiffness tends to reduce both the vertical and lateral movements. The effect is greatest upon the centreline settlements and least upon horizontal movement at the toe of the embankment. Increasing embankment stiffness tends to enhance the role of the geotextile and, thus, increasing fabric stiffness has a slightly greater effect on deformations for a stiff fill than for a loose fill. However, in all cases the effect of fill stiffness upon deformations is relatively small (ie. typically less than 12%). It was also found that the fill stiffness (assuming the same strength parameters) had negligible effect upon the stability of the geotextile reinforced embankment.

CONCLUSIONS

A technique for the analysis of geotextile reinforced embankments was used to examine the factors affecting the performance of an embankment constructed at Pinto Pass, Alabama. The analysis permits consideration of the development of membrane forces, variable foundation support characteristics, internal embankment arching and load redistribution, slip at the soil-fabric interface and simulation of complex construction sequences. The approach allows the determination of embankment deformations, membrane forces and stability.

The results of this theoretical study indicate that for this case:

1. The outside-inside construction technique proposed by Haliburton et al (7) is superior to normal construction in horizontal lifts even without the use of fabric; however, the role of the fabric may also be enhanced by the technique, particularly for stiff fabrics.
2. The fabric at the edge of the embankment was predicted to be unstressed. Thus, sufficient fabric anchorage could be mobilized without the expense and inconvenience of overlapping of the fabric at the edge of the embankment.
3. The fabric has relatively little effect upon vertical settlements (provided collapse does not occur) but may significantly reduce lateral spreading. // again
4. For this case, the displacements varied approximately linearly with the logarithm of fabric stiffness. (Additional research is required before the generality of this relationship could be accepted).
5. The fabric does increase the stability of the embankment. However, for fabric with low to moderate stiffness, extremely large deformations may occur prior to the fabric reaching its tensile capacity, and in these cases, failure may be deemed to have occurred prior to rupture of the fabric.
6. A knowledge of <u>both</u> the fabric stiffness and tensile capacity, determined under condition of plane strain, is considered to be essential for the analysis and design of geotextile reinforced embankments.
7. Although some consideration must be given to the fill stiffness, it would appear that for the range of cases considered herein, a precise determination of the fill stiffness is unnecessary. //

ACKNOWLEDGEMENT

The work described in this paper forms part of a general study into the behaviour of geotextile reinforced embankments being funded by the Ontario Ministry of Transportation and Communications. Additional funding was supplied by the Natural Science and Engineering Research Council of Canada under Grant No. A1007. The assistance of Mr. T. Ko and Mr. G. Lusk in the preparation of the figures is also gratefully acknowledged.

REFERENCES

(1) Carter, J.P., Booker, J.R. and Small, J.C. (1979). "The analysis of finite elasto-plastic consolidation", Int. J. Num. and Analytical Methods in Geom., Vol. 3, No. 2, pp. 107-130.

(2) Davis, E.H. (1968). "Theories of plasticity and failure of soil masses", Chapter 6 "Soil Mechanics - Selected Topics", Ed. I.K. Lee, Butterworths.

(3) Fisher, D.G. (1982). "Foundations on non-homogeneous soils", M.E.Sc., University of Western Ontario (In preparation).

(4) Fowler, J. (1979). "Analysis of fabric-reinforced embankment test section at Pinto Pass, Mobile, Alabama". Ph.D. Thesis, Oklahoma State University.

(5) Haliburton, T.A. (1981). "Use of engineering fabric in road and embankment construction". Seminar on the Use of Synthetic Fabrics in Civil Engineering, Nov. 19, 1981, Toronto, pp. 66-94.

(6) Haliburton, T.A., Angling, C.C. and Lawmaster, J.D. (1979). "Testing of geotechnical fabric for use as a reinforcement". Geotechnical Testing Journal, GTJO DJ, Vol. 1, pp. 203-212.

(7) Haliburton, T.A., Fowler, J. and Langan, J.P. (1980). "Design and construction of fabric reinforced embankment test section at Pinto Pass, Mobile, Alabama". Transportation Research Record, 749, pp. 27-33.

(8) Rowe, R.K., Booker, J.R. and Balaam, N.P. (1978). "Application of the initial stress method to soil-structure interaction". International Journal for Numerical Methods in Engineering, Vol. 12, No. 5, pp. 873-880.

Most important fact for designing and dimensioning is elastic equilibrium