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The Behavior of Geotextile Reinforced Clay Subject to Undrained Loading

Le comportement de l'argile renforcée au géotextile soumise à une charge non asséchée

In the rapid loading of clay soils there is often no opportunity for dissipation of the porewater pressure generated. For reinforced clay this short term, or undrained condition might be associated with instability. Amongst other things the response of reinforced clay to undrained loading will be a function of the nature of the reinforcement. To investigate possible responses three different geotextiles were installed in cylindrical clay samples subject to either rapid shear or shear at constant volume, both of these regimes being consistent with the notion of undrained loading. It was found that even permeable geotextile reinforcement caused a consistent and substantial decrease in strength compared to that of an unreinforced sample, however, as reinforcement spacing decreased the strength of the soil improved and ultimately exceeded that of the unreinforced clay. This response reflects the fact that rapid loading is not necessarily associated with undrained loading if reinforcement, which also acts as a drain, is installed at a sufficiently small spacing.

Dans la charge rapide des sols argileux il n'y a souvent aucune chance que l'eau produite par la pression se dissipe.Pour l'argile reinforcée cette courte période,ou état de non assèchement peut être associée à l'instabilité.La réaction de l'argile renforcée à une charge non asséchée sera fonction,entre,autres choses,de la nature du renforcement. Pour examiner les réactions possibles trois géotextiles différents ont été installés dans des échantillons d'argile cylindriques soumis soit à une déformation rapide soit à une déformation constante en volume,ces deux régimes étant conformes à la notion de charge non asséchée.On a trouvé que même le renforcement géotextile perméable causait une diminution régulière et substantielle de la résistance par rapport à un échantillon non renforcé;cependant,avec la réduction de l'espacement du renforcement la résistance du sol s'améliorait . et finalement dépassait celle de l'argile non renforcée.Cette réaction fait ressortir le fait que la charge rapide n'est pas nécessairement associée à la charge non asséchée si le renforcement,qui agit en facteur de drainage,est mis en place avec un espacement suffisamment réduit.

INTRODUCTION

Reinforced soil exhibits several advantages not least of which is speed of construction. In the case of free draining granular backfill rate of construction is of little consequence since even for rapid loading the fully drained condition may be deemed to prevail. This is not necessarily the case for low permeability cohesive fill where rapid construction is likely to be associated with undrained loading. To investigate the possible effects of rapid loading several series of triaxial compression tests were carried out on cylindrical samples of clay reinforced with discs of various geotextiles. Three geotextiles were used namely porous sintered polythene, a thick needle punched felt, and a melt-bonded fabric. Three clay soils were used, however, these all had very similar strength, deformation and consolidation characteristics.

A simple test procedure was followed involving the determination of the strength of the unreinforced clay and the apparent strength of the same clay reinforced with geotextile discs at various spacings. The change in strength caused by the reinforcement is quantified by the introduction of a strength ratio which is simply the measured deviator stress at failure in the reinforced sample divided by the deviator stress at failure in the unreinforced sample. By varying the spacing of a given geotextile in a given soil type it is possible to observe the effect of this parameter on strength ratio. Since all the clays employed exhibited similar properties it is possible to make valid comparisons of the effects of different reinforcing materials. All tests were carried out under conditions of undrained shear,however several of the tests were sheared at a rate consistent with porewater pressure equalisation thus permitting measurement of porewater pressure.

1. SOIL AND REINFORCEMENT PROPERTIES

Three clay soils were used in the investigation,Kaolin clay, boulder clay and London clay. All of these clays were thoroughly remoulded, following which cylindrical samples were formed using a hydraulic press. This process was followed by saturation under back pressure and subsequent consolidation under an isotropic stress regime. The relevant properties of the three clays are summarised in Table 1 which shows the plasticity and effective strength parameters to be similar.

To give some indication of compressibility initial undrained deformation moduli are also given in Table 1. Since the deformation modulus is, amongst other things,a function of consolidation pressure a range of moduli and corresponding consolidation pressures is indicated. A range of geotextile reinforcement was employed as indicated in Table 2.

All of the geotextiles used have permeability, normal to the plane of the material, several orders of magnitude higher than those of the clay. Due to the structures of the synthetic reinforcement it is not possible to define a deformation modulus in the strict sense,however, to permit some comparison with moduli for the soils, Table 2, gives values for a notional initial modulus. This is derived from plane strain tensile test data and the nominal thickness of the structure which allows calculation of a notional stressed area. The structure thickness, especially in the non-woven fabric and the

Table 1. Soil Data.

Soil	Liquid Limit %	Plastic Limit %	ϕ'	Initial Modulus MN/m^2	Consolidation Pressure kN/m^2
Kaolin	57	33	24.3	13	250
Remoulded London Clay	77	31	19.8	7-28	100-400
Remoulded Boulder Clay	68	39	20.5	7-28	50-200

Table 2. Reinforcement Data.

Reinforcement	Material	Structure	Weight g/m^2	Thickness mm	Initial Modulus MN/m^2		Tensile Strength kN/m	
					Min.	Max.	Min.	Max.
Porous Plastic	Polythene	Sintered	3280	4.75	75	110	18.0	18.6
Non-woven Fabric	Polypropylene/ Nylon	Melt Bonded	280	0.70	30	42	8.1	9.0
Felt	Polypropylene	Needle Punched	250	3.20	2	7	5.5	10.9

felt will tend to decrease due to confinement by the soil and tensile loading of the reinforcement. Thus the values given in Table 2 would represent a lower bound. Comparison of the soil and reinforcement moduli show that with the possible exception of the felt the reinforcement deformation moduli are higher than those for the soil.

2. TESTS AND TEST RESULTS

A total of four series of tests are reported. Sample preparation involved thoroughly remoulding the clay, followed by hydraulic pressing of standard 204mmx102mm diameter samples at moisture contents and dry densities theoretically consistent with full saturation. All reinforcement was cut to form discs having the same diameter as the soil sample. To allow introduction of the reinforcement each sample was cut into a number of thick discs, of equal height, using a cheese wire. Each soil sample was then reassembled with a disc of reinforcing material being introduced between each soil disc so producing a multi-reinforced sample. All test results are reported in the same format namely the failure strength ratio, R, and inverse aspect ratio μ. The strength ratio is defined as the ratio of measured compressive strength, or failure deviator stress, of the reinforced sample to that of an unreinforced sample with the latter being either measured directly for an unreinforced control sample or derived from some other measured soil property which had been previously related to the unreinforced strength. The inverse aspect ratio is simply defined as d/h where d is the sample diameter and h is the height of an individual soil cell within the multi-reinforced sample.

In the first series of tests the effects of a permeable reinforcement were investigated using a porous plastic reinforcement within multi-reinforced 102mm diameter Kaolin samples. All samples were sheared unconfined at a rate of strain of 2% per minute with unreinforced undrained shear strengths being determined from a previously derived moisture content-undrained shear strength relationship, Ingold (1). On this occasion eight pairs of reinforced samples were tested with a wide range of inverse aspect ratios. The results, which are summarised in Table 3, show strength ratios falling below unity for small inverse aspect ratios. To widen the scope of the investigation the next two series of tests were carried out using multi-stage consolidated undrained tests with porewater pressure measurement on remoulded boulder clay reinforced with geotextiles in the form of a non-woven fabric, Test Series No. 2, and a needle punched felt, Test Series No.3. Results for these two series are given in Table 3. Sample preparation methods for the final test series deviated considerably from those previously described. Although use was again made of Kaolin clay and porous plastic reinforcement, the clay was consolidated from a slurry to produce an isotropic sample whose unreinforced strength could be related to effective consolidation pressure. In place of the multi-reinforced sample a single unit cell of soil was employed this being bound top and bottom by a porous plastic reinforcing disc. To ensure compatibility of stress regimes developed within unit cell and multi-reinforced samples it was necessary to minimise friction between the outer faces of the reinforcing discs and the

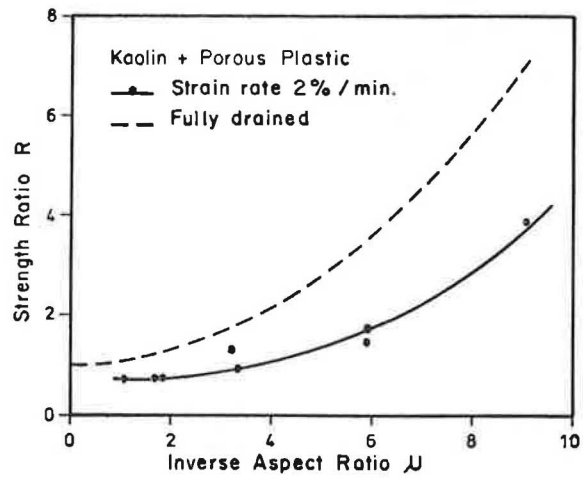

Fig. 1. Results of Test Series No. 1.

Table 3. Summary of Test Results.

Test Series	Test No.	Test Type	Reinforcement	Soil	Inverse Aspect Ratio	Strength Ratio
1	1	UCUD*	Porous Plastic	Kaolin	1.09	0.71
	2	"	" "	"	1.72	0.79
	3	"	" "	"	1.79	0.79
	4	"	" "	"	3.23	1.30
	5	"	" "	"	3.33	0.94
	6	"	" "	"	5.88	1.45
	7	"	" "	"	5.88	1.70
	8	"	" "	"	9.09	3.92
2	9a	CUD+PWP+	Non-woven Fabric	Remoulded Boulder Clay	4.17	1.08
	9b	"	" "	"	4.43	1.15
	9c	"	" "	"	4.72	1.17
3	10a	"	Felt	"	4.29	1.22
	10b	"	"	"	4.74	1.30
	10c	"	"	"	5.18	1.32
	11a	"	"	"	4.30	1.13
	11b	"	"	"	4.74	1.18
	11c	"	"	"	5.18	1.23
4	12	CUD+PWP°	Porous Plastic	Kaolin	3.41	1.57
	13	"	" "	"	4.65	1.98
	14	"	" "	"	6.99	2.50
	15	"	" "	"	13.89	3.21

* Unconsolidated unconfined undrained tests - strain rate 2% per minute.

\+ Multi-reinforced multi-stage consolidated - undrained with porewater pressure measurement.

° Unit reinforced cell consolidated - undrained with porewater pressure measurement.

pedestal and top cap of the triaxial apparatus. This was achieved by the introduction of thin lubricated rubber end plattens, the lower platten being provided with a central opening to allow the measurement of porewater pressure via the conventional porous stone mounted on the pedestal. Using this arrangement four consolidated undrained tests with porewater pressure measurement were carried out at different inverse aspect ratios with the results being summarised in Table 3, Test Series No. 4.

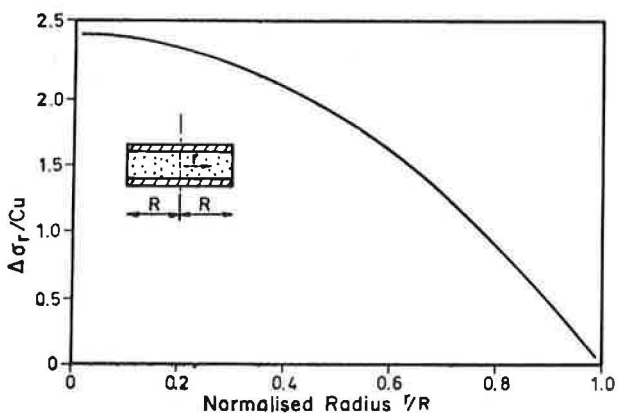

Fig. 2. Radial Variation of $\Delta\sigma_r/Cu$

3. ANALYSIS OF TEST RESULTS

The results of Test Series No. 1, which are plotted in Figure 1, were analysed using a second order regression analysis which rendered a coefficient of correlation of 0.983 . This is shown as a solid line in Figure 1 together with the results of a similar analysis, shown in broken line, for fully drained test results reported elsewhere, Ingold (1). Reference to the solid line shows the strength ratio dropping below unity for inverse aspect ratios less than approximately 4. Since such values of μ are associated with long vertical drainage paths the rapid rate of shear of 2% per minute is associated with truly undrained loading in which case premature failure is induced. It can be shown from earlier work, Yang (2), Ingold (3), that the introduction of reinforcement leads to an enhancement of radial total stress, $\Delta\sigma_r$, and a corresponding increase in global total stress $\Delta\sigma_3$. For a saturated soil this would be associated with a change in porewater pressure of equal magnitude and therefore no change in effective stress, Skempton (4). If, however, consideration is given to the radial distribution of $\Delta\sigma_r$ and therefore by implication $\Delta\sigma_3$ it can be seen that this is not uniform with $\Delta\sigma_r$ decreasing radially, Figure 2. As a consequence of this the highest porewater pressures, which are continuously generated near the centre of the sample during the initial stages of loading, migrate radially so causing a net decrease in minor principal effective stress throughout the sample. For a soil obeying the Mohr-Coulomb failure criterion a change in minor principal effective stress is associated with a change in strength. Since in this case the minor principal stress decreases there would be a corresponding decrease in strength. For values of $\mu>4$ the vertical drainage paths

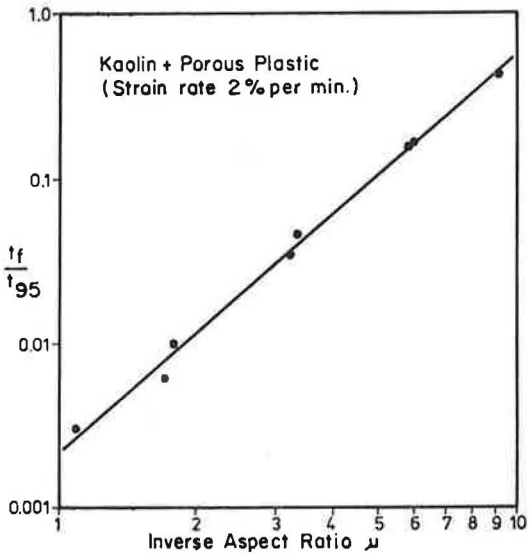

Fig. 3. Variation of t_f/t_{95} with μ.

become shorter in which case even this rapid rate of shear is associated with partial drainage. In consequence of this the deleterious porewater pressures associated with an increase in total stress, $\Delta\sigma_3$, are allowed to partially dissipate in which case there is an increase in effective stress, $\Delta\sigma_3'$, leading to an enhanced strength. For the highest value of $\mu \cong 9$, the regression analysis is almost parallel to that for the fully drained condition shown by the broken line in Figure 1. That increasing μ is associated with increasing degree of drainage becomes evident from Figure 3 which shows a plot of actual failure time, t_f, divided by theoretical failure time for 95% drainage, t_{95}, as a function of μ. As can be seen values of t_f/t_{95} approach unity, that is the fully drained condition as μ becomes progressively larger. The logical implication of this is that if porous reinforcement is installed at sufficiently close centres then fully drained conditions can be made to prevail under rates of loading, or construction, that would normally be associated with undrained loading.

The effects of other geotextile reinforcements were investigated in Test Series Nos.2 and 3. The results are given in Table 3 which shows strength ratios varying between 1.08 and 1.32 for inverse aspect ratios in the range 4.17 to 5.18. Remembering that these tests were executed under truly undrained conditions it is remarkable that there was any strength increase at all since an improvement in strength must be associated with an increase in minor principal effective stress, $\Delta\sigma_3'$. To explore this phenomenon further Test Series No. 4 was carried out using Kaolin clay, consolidated from a slurry and the porous plastic reinforcement employed in earlier tests. As for the non-woven geotextiles the test regime was consolidated-undrained with porewater pressure measurement, however, tests were conducted on unit cells, rather than multi-reinforced cells to avoid any ambiguity in porewater pressure measurement.Results from these tests given in Table 3 confirm that the strength ratio does in fact increase with increasing inverse aspect ratio. To probe the possible mechanism of this strength increase it is necessary to attempt a more detailed analysis of the results.

Earlier radiographic analyses, Ingold (1), indicated that although there is substantial rotation of principal stress axes the measured vertical effective stress in the reinforced sample can be approximated to the major principal effective stress. If it is assumed that failure within the reinforced soil occurs at the same principal effective stress ratio, K, as unreinforced soil, then a mean value of minor principal effective stress, $K\sigma_1'$, can be backfigured from the measured value of major principal stress, σ_1'. The difference between this derived value of mean minor principal stress acting within the sample and that applied at the boundary of the sample σ_3' may be taken to be the mean increase, $\Delta\sigma_3'$, induced by the reinforcement. This condition is represented by equation (1):

$$\Delta\sigma_3' = K\sigma_1' - \sigma_3' \qquad (1)$$

Calculated values for an unreinforced control sample and four reinforced samples are shown in Table 4 together with the corresponding values of inverse aspect ratio μ and strength ratio R.

As can be seen increasing R is associated with increasing $\Delta\sigma_3'$ and μ as expected. What was not expected was that the measured porewater pressure at failure, u, also increased with μ, Table 4. Such an increase in porewater pressure would be expected to be associated with a decrease in strength. Thus the results on cursory examination lead to the apparent inconsistency of an increase in effective stress, $\Delta\sigma_3'$, associated with an enhanced porewater pressure at failure. The only explanation for this is that there must be an increase in total stress, $\Delta\sigma_3$ which is greater than the excess porewater pressure at failure, Δu by an amount $\Delta\sigma_3'$. Inspection of equation (2) shows that this requirement is consistent with the laws of effective stress:

$$\Delta\sigma_3' = \Delta\sigma_3 - \Delta u \qquad (2)$$

A possible mechanism controlling these apparently conflicting requirements can be deduced from Skempton's general porewater pressure equation which can be applied to the shear stage of the reinforced samples, equation (3):

$$A = \frac{u - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3} \qquad (3)$$

The resulting calculated values of the porewater pressure parameter A are shown in Table 4, as the values of the

Table 4. Analysis of Test Results Test Series No. 4.

Test No.	μ	R	$\Delta\sigma_3'$ kN/m²	u kN/m²	Δu kN/m²	$\Delta\sigma_3$ kN/m²	A
Control	0.50	1.00	0	124	0	0	0.70
12	3.41	1.57	20	180	56	76	0.51
13	4.65	1.98	45	192	68	113	0.33
14	6.99	2.50	71	221	98	169	0.19
15	13.89	3.21	118	237	113	231	0.02

porewater parameter that would be required for the corresponding values of $\Delta\sigma_3'$ to be realized. As can be seen these values, which are for the reinforced composite, are lower than the measured A value of 0.70 obtained for the soil alone from the unreinforced control sample. Since the A value of the soil alone may be taken as a constant of the soil for a particular stress history and stress level it follows that any modification of the A value of the reinforced composite must stem from the reinforcement. To investigate this a sample of reinforcement of the same diameter and same mean height as that used in the reinforced soil sample, was tested to measure its A value directly. This was found to be 0.08. A similar measurement for the felt reinforcement rendered a mean value of 0.27 compared to 0.75 for the soil alone. The consequence of these low A values in the reinforcement is that the high porewater pressures generated in the soil by $\Delta\sigma_3$ migrate towards the reinforcement which does not exhibit such a high porewater pressure response. Obviously since the porewater pressure in the body of the soil is reduced there is an increase in the effective stress $\Delta\sigma_3'$ and a corresponding increase in strength ratio. As the reinforcement spacing decreases the magnitude of $\Delta\sigma_3$ and therefore porewater pressure increases. Since the ability of the reinforcement to reduce porewater pressure is limited its effects begin to become suppressed and the strength ratio does not increase at the same rate as it might in a drained loading situation. This is shown clearly in Figure 4 and

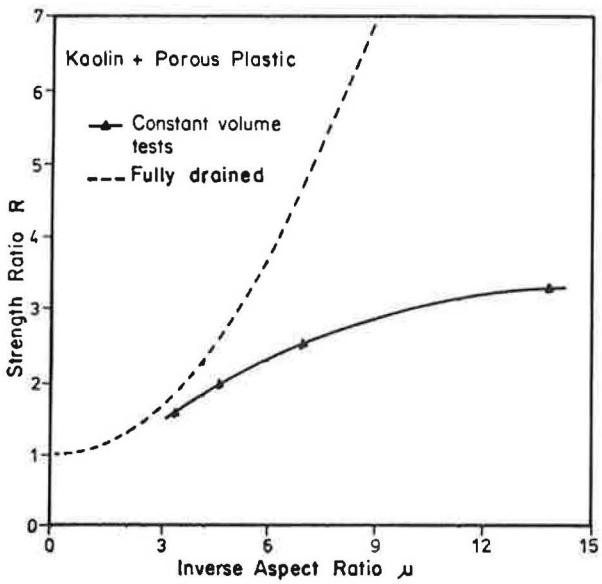

Fig. 4. Results of Test Series No. 4.

follows from Table 4 which shows the "required" A value decreasing with increasing μ. At the highest inverse aspect ratio tested the required A value of 0.02 is slightly less than that of the reinforcement alone therefore the implied increase of $\Delta\sigma_3'$ of 118kN/m² represents the maximum that could ever be developed by the particular reinforcement and test conditions employed.

4. CONCLUSION

A total of fifteen triaxial compression tests were carried out on samples of very similar clay soils using three different reinforcing materials. All the tests were ostensibly sheared undrained either by virtue of rapid rate of shear or, more positively by ensuring shear at constant volume. The phenomenon of strength reduction under rapid loading was observed in clay reinforced with porous plastic, however, this reduction in strength was only observed at large reinforcement spacing where a rapid rate of shearing is consistent with undrained loading. With decreasing reinforcement spacing and consequently decreasing vertical drainage path length there is a partial dissipation of porewater pressure permitting an apparent increase in minor principal effective stress and therefore compressive strength. The logical implication of these results is that permeable reinforcement placed at sufficiently close centres might allow fully drained conditions to prevail at rates of construction normally associated with undrained loading.

On investigating the non-woven fabric and felt reinforced samples under truly undrained conditions it was found that they enhanced compressive strength. This finding appeared anomalous since a gain in strength is associated with an increase in effective stress, however, the previous tests indicated that under undrained conditions there is likely to be a decrease in strength. Further tests, involving unit cells of clay reinforced with porous plastic showed that although there was a high porewater pressure induced this appears to migrate to the reinforcement which exhibits a much lower pore pressure parameter A than the soil alone. In consequence of this there would be a reduction in pore water pressure in the soil resulting in an increase in minor principal effective stress and hence compressive strength. In the particular tests reported the strength enhancement was limited, at high inverse aspect ratios, by the finite ability of the reinforcement to depress induced porewater pressure. Finally it must be remembered that the data presented relates to laboratory tests carried out on saturated clay loaded under an axisymmetric stress regime. In all cases failure was by bond as opposed to tensile failure of the reinforcement which was applied in the form of a continuous disc. These conditions deviate considerably from those likely to prevail on site where the stress regime would normally be plane-strain and the soil partly saturated.

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