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EFFECT OF SAND REINFORCED WITH DISCRETE INCLUSIONS & THE STUDY OF BEHAVIOUR OF MODEL FOOTINGS

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ABSTRACT

A progression of lab model tests on a strip balance upheld by a sand built up by arbitrarily conveyed polypropylene fiber and lattice components was led to contrast the outcomes and those acquired from unreinforced sand and with one another. For leading the model tests, a uniform sand was compacted in the test box at its ideal dampness content and most extreme dry density. Three types of support, two sizes of cross section components having a similar opening size and one size of fiber component cut from the lattices, were utilized in differing sums in the tests. Results demonstrated that support of sand by arbitrarily disseminated considerations caused an expansion in a definitive bearing capacity esteems and the settlement at a definitive burden overall. The adequacy of discrete building up components was seen to rely upon the quantity just as the state of the considerations. The bigger cross section size was observed to be better than different considerations considering a definitive bearing capacity esteems. For the lattice components there has all the earmarks of being an optimum consideration proportion, while strands showed a directly expanding pattern based on an expansion in extreme bearing capacity for the scope of reinforcement amounts utilized.

1 INTRODUCTION

Early soil reinforcement techniques typically consisted of strips, grids or sheets (woven or non-woven geotextiles) placed horizontally in the soil. Later other types of reinforcement, for example continuous filaments analogous to fibrous root reinforcement and randomly distributed small discrete inclusions in the form of fibres, rods, discs, shavings, meshes and multioriented elements, have received increasing attention (Lawton *et al.,* 1993). Of these, the use of continuous polymeric filaments mixed with sandy soil under the proprietary name 'Texsol' (Leflaive, 1985) has developed as an established method in retaining walls and slope protections and already more than 80 successful applications have been reported (Khay er of., 1990; Ishizaki *et ul.,* 1992).

Several studies aimed at examining the mechanics of soil reinforcement by discrete inclusions and at exploring the feasibility of using them in practical applications have been reported. In these studies direct shear, triaxial compression, plane strain triaxial, **CBR,** model footing tests and also dynamic tests were performed using various types of fibres in the majority of cases. Improvement in engineering properties of granular soils by the incor-poration of discrete inclusions in the soil is reported with a few exceptions. For instance, use of very short, thin steel fibres resulted in decreased static strength compared to unreinforced soil at the same density (Verma & Char, 1978). Hoare (1979) presents results of compaction and triaxial compression tests on dry, angular crushed sandy gravel mixed with polymeric fibres and small strips cut from a geotextile. He concludes that reinforcement offers resistance to compaction and reinforcement has beneficial effects on both the strength and the ductility, except when the increased amount of reinforce-ment results in reduced density, in which case the strength may even decrease.

Direct shear test results on a dry sand with different types of fibres extending at various angles over the shear plane showed that reinforcement increased the peak shear strength and limited post peak reductions in shear resistance (Gray & Ohashi, 1983). Gray and Al-Refeai (1986) report the results of comparison triaxial compression tests on the stress—deformation response of a dry sand reinforced with continuous, oriented geotextile layers as opposed to discrete, randomly distributed fibres. They concluded that fibre reinforcement increased both the ultimate strength and the stiffness. The decrease in stiffness at low strains observed with geotextile inclusions did not occur with the fibres. The increase in strength with fibre content varied linearly up to a fibre content of 2% by weight, and thereafter approached an asymptotic upper limit. Ranjan *et al.* (1994) also cite an upper limit of 2% by weight. Maher and Gray (1990) describe the effect of the sand properties (i.e. gradation and particle size and shape) and fibre properties (i.e. weight frac- tion, aspect ratio and modulus) on the strength and deformation behaviour of sand reinforced with randomly distributed fibres and suggest that the asymptotic upper limit of fibre content is mainly governed by the confining stress and fibre aspect ratio. The existence of such an upper limit is also observed in dynamic tests carried out on the same composites (Maher & Woods, 1990). Both shear modulus and damping increased approximately linearly with increasing amount of fibre to about 4%, then tended to approach an asymptotic upper limit at approximately 5% fibre content by weight. Contrary to these observations, special large direct shear test results on a dry sand reinforced with fibres, wood doweling and metal rods did not yield a linear relationship between reinforcement concentration and increased strength (Shewbridge & Sitar, 1989).

Direct shear test results on a uniform silty sand reinforced with aligned and randomly orientated metallic fibres of varying flexibility are discussed by Fatani *et al.* (1991). Vertically placed fibres were found to be most effective. Dynamic responses for sands reinforced with randomly distributed fibres and with fibres orientated vertically to the shear plane were found to be similar (Maher & Woods, 1990). Noorany and Uzdavines (1987), who performed cyclic triaxial tests on sand with four different reinforcing elements in nine different configurations, concluded that the specimens with randomly distributed fibres exhibit relatively higher resistance to liquefaction. Triaxial tests performed on two sands reinforced with glass fibre and mesh elements examined the effect of soil type, extensibility and shape of inclusions and fibre length, and led to the conclusion that mesh elements were superior to fibres, especially in the case of fine sand (Al-Refeai, 1991). Al Refeai's results on the stiffness at low strains were mixed; in some cases the reinforced soils were stiffer at all strains but in other cases the reinforced soils were less stiff at small strains, as also found by Freitag (1986) based on the data from unconfined compression tests on reinforced compacted fine-grained soil.

More recently the use of mesh elements (Mercer *et al.,* 1984; McGown *et al.,* 1985; Andrawes *et al.,* 1986) in soil strengthening has been the subject of some research. Meshes are believed to have an additional 'interlocking' mechanism in reinforcing. Mercer *et al.* (1984) and McGown *et al.* (1985) report the results of laboratory compaction, CBR, triaxial compression and model footing tests on sand reinforced with 40 and 50 mm square poly- propylene meshes of the same kind as the ones used in the present investi- gation and compacted around optimum moisture content. The compaction test results indicate that resistance to compaction is significant for mesh contents in excess of 0-6% by dry weight. Peak deviator stresses and bearing pressures at any strain level, i.e. stiffness at small strains, as well as CBR values increase due to mesh reinforcement. Field trials suggest that mesh contents of between 0 1 and 0-2% will be suitable (Andrawes *et al.,* 1986). Uysal (1993) performed comparison triaxial tests on a sand reinforced with the 30 mm x 50 mm size of the same type of meshes used in the previously mentioned investigations and the present study, and also with 5-mm long fibres cut from the meshes. The sand was compacted at the optimum moist- ure content and inclusion contents of 0-10, 0 20 and 0 30% were used. The increase in peak deviator stress varied linearly over 6-59%, depending on the type and amount of inclusions as well as the confining pressures: the increase was significantly larger for smaller confining pressures. Failure strains of fibre-reinforced sand samples were generally lower than the failure strains for mesh-reinforced samples. No clear superiority of mesh elements over fibres was observed.

The present study examines the load—settlement data from model strip footing tests on unreinforced sand and on sand reinforced with two sizes of the same type of mesh elements used in the previous investigations and one size of fibre element cut from the meshes. The main objective was to assess the relative reinforcing efficiency of mesh and fibre elements at the same inclusion ratio. It was also thought useful to check if the improvements in soil strength observed in the triaxial compression tests by Uysal (1993) are comparable to the results of model footing tests.

2 EXPERIMENTAL PROCEDURE

The general layout of the equipment used in the present study is illustrated in Fig. 1. The model footing was made out of steel plate of 20 mm thickness and measured 50 mm (width) x 250 mm (length). Use of a larger model was precluded by the limitations of the loading system. It had a smooth bottom face and a hole at the centre of the top face for mounting the proving ring.

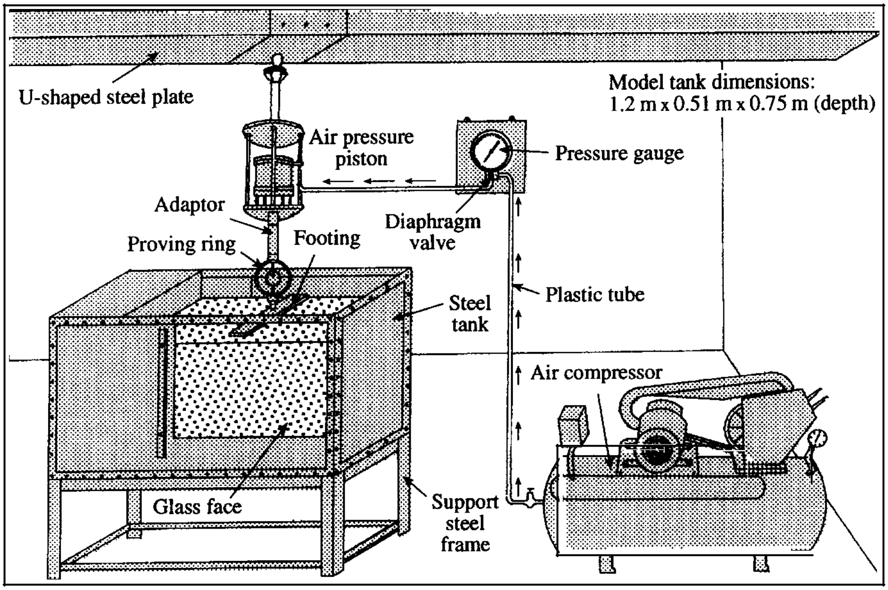

Fig. 1. Testing equipment.

Two dial gauges were attached along the centre-line on both sides and near the ends of the footing to measure the displacement (settlement).

A uniform medium sand whose properties are given in Table 1 was used in the experiments. The sand was placed in the box by compacting at the optimum moisture content to its maximum dry density. Allowance was made for the volume occupied by the inclusions in tests on reinforced sand. The shear strength parameters for the compacted sand in the low normal stress range of 20—70 kPa were determined using cylindrical wedge shear tests (Mirata, 1991) which produce stress conditions closer to those of a plane strain test. The shear strength parameters for a very similar sand measured by triaxial tests using cell pressures 50 kPa and larger are c' — 20 kPa and Q' = 36 5° (Uysal, 1993).

The polypropylene mesh reinforcements supplied by Netlon Ltd, Blackburn, UK, were in two ready cut sizes of 30 x 50 mm and 50 x 100 mm with the same opening size of 10 x 10 mm. They will be called the 'small mesh' and the 'big mesh' elements, respectively, in the text. Fifty-millimetre-long fibres were produced by cutting the meshes, as a result of which each fibre had 4-5 knots over its length and the existence of these knots is believed to enhance the reinforcing effect of the fibres.

For conducting the model tests moist sand/reinforced sand was compacted in the test box in layers of 50 mm thickness. It was attempted to achieve a uniform distribution of reinforcing inclusions by the use of a special guide grid. The model footing was placed on the surface of the compacted sandy reinforced sand bed. The load was applied by air pressure supplied by an air-pressure piston in a stress-controlled manner. The load and the corresponding foundation settlement were measured by the proving ring and the two dial gauges placed on each side of the centre-line of the footing. The details of the experimental work are presented elsewhere (Biitiin, 1995).

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TABLE 1

Properties of the Sand Used in the Model Footing Tests

Parameter	*Quantit y*
Uniformity coefficient, *Cq*	3 995
Coefficient of curvature, C,	1 132
Effective size, *D* (mm)	0 205
Duo (>>)	0-436
Dg (mm)	0 819
Specific gravity of soil solids, G,	2-587
Maximum dry density (Mg/m ^{3})	1 724
Optimum water content (%)	11 3
Cohesion, c' (kPa)	6-98
Angle of shearing resistance, ' (degrees)	47 8

3 TEST RESULTS

Tests on unreinforced sand and sand reinforced with one size of fibre and two sizes of mesh were performed at inclusion ratios of 0-075, 0-10 and 0-15% by dry weight, which are comparable to the suggested mesh reinforcement contents of 0-1H-0-20% for practical applications. The number of tests carried out was 18 including the repeat tests. Pressure—settlement data were evaluated to produce the best fitting polynomial curve. For multiple tests an average curve was obtained. Figure 2 presents the pressure—settlement curves or average curves thus obtained.

The examination of the figure shows that all the curves except two lie above the curve for the unreinforced sand, indicating the improvement brought about by the reinforcements. This is especially pronounced at large displacements and for the larger size meshes used in the experiments. It is

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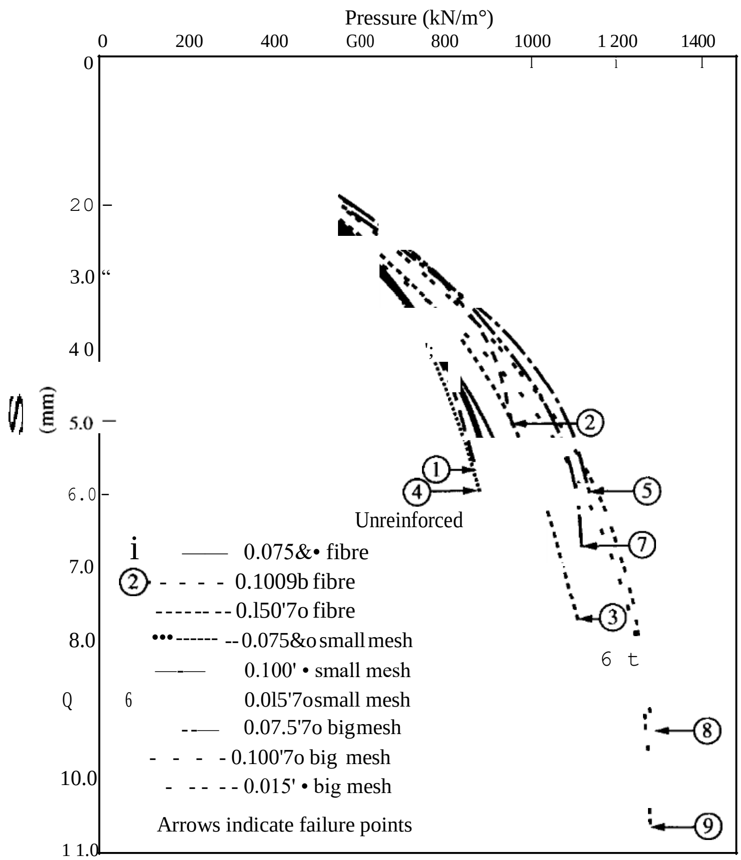

Fig. 2. Bearing-pressure—settlement curve.

also observed that for the big meshes failure takes place at much larger settlements. The curves that lie below the one corresponding to the unreinforced case at large settlements are for the lowest reinforcement percentage and for the small meshes and the fibres. An enlargement of the initial part of Fig. 2 corresponding to loads smaller than half the failure load for the unreinforced case indicates that most of the curves lie very slightly above the one for the unreinforced case (i.e. the improvement is not significant) and no noticeable or consistent trend regarding the effect of type and amount of different inclusions is observed at small settlements (Bittun, 1995).

The effect of the inclusion shape as well as the inclusion quantity on the failure load (i.e. the ultimate bearing capacity) is illustrated in Fig. 3. Here the BCRS is the bearing capacity ratio defined as the ratio of ultimate bearing capacity of the reinforced soil to that of unreinforced soil. The figure quantifies the previous observations as given in Fig. 2 as well as allowing variation trends to be established. For the quantities and shapes of the discrete reinforcements used in the experiments the change in the ultimate bearing capacity lies between about 40% and —5%: the higher value is for the big meshes at inclusion ratios of 0-10 and 0-15% and the negative value is for the fibres and the small meshes at the lowest inclusion ratio of 0 075 %. At all inclusion ratios the use of the big meshes brings about the greatest improvement compared to the others. Although the trials at only three reinforcement percentages do not allow definite statements to be made, the plots in Fig. 3 show that for both the meshes the BCR, values increase sharply by comparable amounts as the reinforcement percentage is increased from 0-075 to 0 10%. At an inclusion ratio of 0-15% the BCR, value remains the same for the big meshes and drops somewhat for the small meshes, suggesting the existence of a possible optimum amount of inclusion quantity for the mesh elements between 0-10 and 0 15%. Unlike the meshes, the BCR, values for the fibres increase linearly as the inclusion ratio increases. It is reasonable to assume

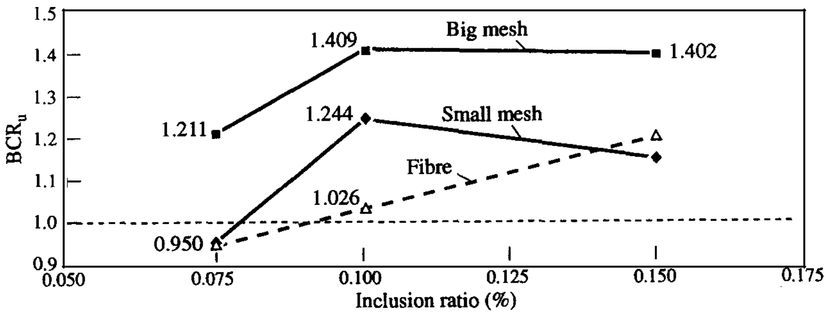

Fig. 3. Variation of BCR, values with inclusion ratio.

that the BCR, value for the fibres will approach an asymptotic upper limit as observed by Gray and Al-Refeai (1986); Maher and Woods (1990) and Ranjan *et al.* (1994).

The displacement or settlement at failure for the sand with and without inclusions is given in Table 2. As can be seen from the Table, the settlement values at failure for the reinforced cases ($S_u(R)$) vary between 0 8 and 1 7 Api u being the settlement at failure for the unreinforced case. This compares well with the results of model strip footing tests on geogrid-reinforced sand where s R) values twice as large as those obtained from the test with unreinforced sand have been found (Khing *et al.*, 1993). Values in the Table suggest that the settlement at failure increases as the inclusion amount increases. This increase is also seen to be dependent on the type of reinforcement: the largest one corresponding to the big meshes and the smallest to the fibres. Uysal (1993) also found that failure strains in triaxial compression tests for the fibre-reinforced sand samples were generally lower than the ones for the small-mesh-reinforced samples; in both cases the reinforcements were identical to those of the present study.

Since limited settlement is generally the design criterion for actual foundations on sand, comparison of the load-bearing values at some selected settlement levels ($s < S_u$) for the reinforced and unreinforced cases was also made, as suggested by Khing e/ *al.* (1993). There is a large scatter of results but the following values may be cited for the big meshes: for the three reinforcement percentages the average increase in the load-bearing value due to inclusions at *s* — 0 50 S_u and s = 0-75 A_u is about 14 and 20%, respectively, as opposed to 34% at *s* — $s_u(R)$ •

4 CONCLUSIONS

Based on the results of the model footing tests performed, the following conclusions can be drawn:

TABLE 2
Settlement at Failure for the Unreinforced and Reinforced Cases

Reinforcement type	*Settlement at failure* és p (*mm*)		
	/J 07S% *inclusion*	*0·10% inclusion*	*0 15°Xa inclu.sion*
Big mesh	6-80	9 31	10 75
Small mesh	5 90	6 01	8 30
Fibre	5 75	5 00	7 78
None		*{s* — 6 40)	

1. The results based on such small-scale load-bearing tests should probably be considered as indicative of penetration resistance and as providing a manner of evaluating the level of soil improvement produced by different inclusions, rather than representative of the field performance of foundations on reinforced soil. The feasibility of constructing footings on granular soils reinforced with randomly distributed discrete inclusions should be established using large-scale models or by testing full-scale trial footings. In particular, the findings of the present model tests described above the other studies relating to less improvement at small settlements (i.e. small increase in stiffness at small strains) have to be checked, since in practice most shallow foundations are designed for limited settlement.
2. The effectiveness of randomly distributed discrete reinforcing inclusions in improving the properties of sands depends on the quantity as well as the shape of the inclusions. Even the overall dimensions of otherwise exactly the same mesh elements have been found to influence the behaviour. Meshes of 50 mm x 100 mm are superior to 50-mm-long fibres cut from these meshes as well as 30 mm x 50 mm meshes at the three inclusion ratios tried for the sand used in the present study. In the case of large-scale applications, trials by performing plate loading tests may be required or a simple test may be devised, e.g. in the form of measuring the imprint dimensions or the penetration depth of a falling object.

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