7 (1991) ...s... 7

GROUND IMPROVEMENT WITH GEOGRIDS

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SYNOPSIS

Results of triaxial tests conducted on large diameter specimens of granular soil reinforced with micro-mesh and circular discs are presented. Based upon analysis of test results, the changes in bearing capacity and settlement have been computed for a footing resting on soft clay overlaid with reinforced sand. The results clearly demonstrate significant improvement in BCR and that improvement if any, in reduction of settlement is marginal.

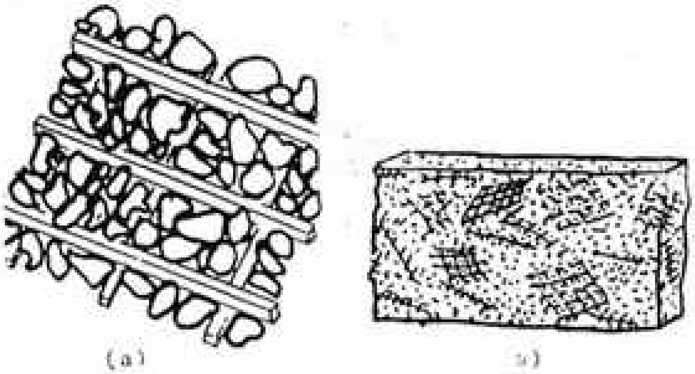
INTRODUCTION

Several research workers have studied the problems of shallow foundation improvement, viz., soil strengthening by root, rope fiber materials man-made fibers, metal strips. Many researchers also studying the effect of a geotextile layer beneath the sand layer over clay bed in a shallow foundation have found that the bearing capacity improved.

In addition, Yamanouchi (1972) studied the improvement of bearing capacity of soft clay by overlaying a granular fill, interfaced with a geomesh. Hyde and Yasuhara (1986) using geogrid (Netlon) as separator between rounded aggregate and an underlying soft clay have found that the settlement reduces and the bearing capacity increases.

The placement of geogrid in the "Localised form" i.e. as layer below conventional foundation may not be an attractive solution. However, if it could be well distributed in micro-mesh form the behaviour of the foundation soil tends to be remarkably different and may find easy application. Recent research in the use of randomly distributed polymeric mesh elements, in soil by Mercer et al. (1984), McGown et al. (1985) and Andrawes et al., (1986) have shown that polymeric (Polypropylene) materials can be placed in a soil to create a composite material with improved stress resistance. Also the meshes interlock with the soil particles produce a strengthening at the mega-scale. The principal advantage of using mesh elements is their interlock action. This occurs at two levels with ribs of individual mesh elements interlocking with groups of soil particles to form an aggregation of particles (Fig.1a), then adjacent aggregation interlocking to form a coherent matrix (Fig.1b).

An attempt has been made herein to understand the bearing capacity and settlement aspects of footing resting on micro-mesh reinforced sand as well as a single layer of geogrid reinforced granular material over clay bed based on experimental data.



(a) (b)

Fig.1 Interlock mechanism for mesh elements between (a) groups of particles (b)adjacent aggregations

EXPERIMENTAL WORK

Materials Used

Soil: The soil used is stone dust, comprising of sub-angular particles, 0.04 to 4.0 mm diameter with a uniformity coefficient of 3.3.

Mesh Elements: The geogrid CE121 manufactured by Netlon India has been used. The physical and mechanical properties of the grid are given in Table 1. Preliminary tests were carried out on different sizes of mesh elements, amongst these 50 mm square elements were considered suitable.

Table 1: Properties of Geogrid

Mesh Opening Size	8x6 mm
Thickness	3.3 mm
Polymer	Polyethylene
Mass per unit area	730 g/m^2
Tensile Strength	7.68 kN/m
Extension at max. load	20.2 %

Triaxial Tests

Saturated stone dust alone and stone dust mixed with various percentages upto 1.4 (by weight) of mesh element were placed in layers into a split mould to obtain 100 mm dia and 200 m high specimens with a dry density of 17.8 kN/m^3. Each layer was compacted to achieve required density by vibration at constant frequency.

Cylindrical specimens have also been prepared with stone dust at the same density and reinforced with circular discs of 100 mm diameter of the geogrid. The number of discs used were 1,2, and 7 equally spaced. Conventional consolidated drained triaxial tests were conducted on the specimens with cell pressures of 25, 50, 100 and 200 kPa at a deformation rate of 0.2 mm/minute.

TEST RESULTS AND DISCUSSION

Micro-Mesh Reinforcement: Figure 2 shows the relationship between deviator stress and axial strain for the stone dust with and without mesh element at different confining pressure. It clearly shows that the mesh increased the deviator stress developed at any strain level which confirms the ability of the mesh element to strengthen the soils. The peak stresses in the granular-mesh mixture occurred at slightly higher axial strain than the sand alone at lower cell pressure. The deviator stress seems to be linearly improving with micro-mesh content.

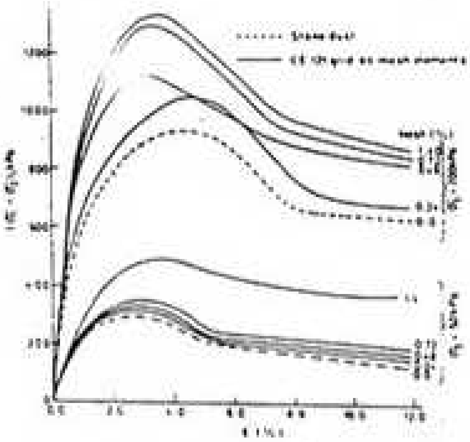


Fig.2 Stress-strain curves for micro-mesh reinforced stone dust for different σ_3

Circular Disc Reinforcement: Figure 3, presents stress-strain curves obtained with disc reinforced triaxial specimens at confining pressures of 50 and 200 kPa. The improvement in strength with reinforcement is amply clear at both the confining pressures, being more marked at the lower confining pressure.

A Comparison: A comparison of the behaviour with disc and micro-mesh reinforced specimens is presented in Fig.4. It may be observed that the improvement in strength is marked at the low confining pressure of 50 kPa for disc reinforced case. This improvement decreases at the pressure of 100 kPa and the strength is nearly the same at confining pressure of 200 kPa. It is also seen that the micro-mesh reinforced specimen is more stiff than the disc reinforced one.

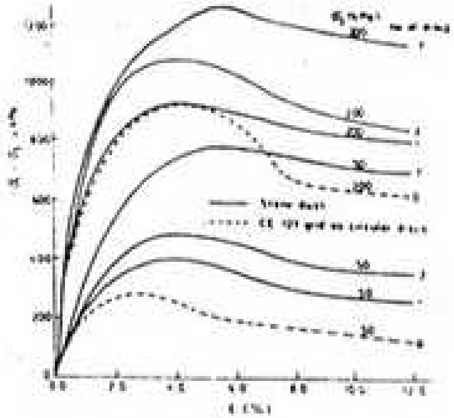


Fig.3 Stress-strain curves for geogrid reinforced stone dust

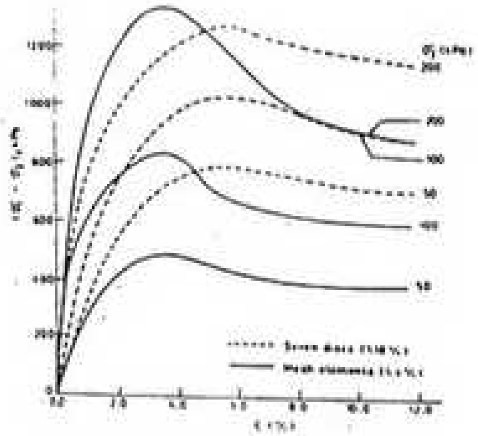


Fig.4 Comparison of stress-strain curves for disc and micro-mesh reinforced stone dust

It should however be borne in mind that the results for disc reinforced specimen cannot be directly utilised for design purposes, as it requires a discrete approach (Raju and Rao 1990). On the other hand one can use the results of the micro-mesh reinforced specimen in design by using the global approach.

Strength Parameters c' and ϕ': The p-q plots for micro-reinforced stone dust and disc reinforced specimens are presented in Fig.5 and Fig.6 respectively.

The failure envelopes are observed to be bilinear for both types of reinforcement which confirm the work reported by Andrawes et al. (1986) for mesh elements. These figures indicate that the initial linear portion is upto a confining pressure of 100 kPa. The values c' and ϕ' obtained are presented in Table 2 and 3. A study of these tables reveals the following:

- For unreinforced stone dust the value of c' is zero and the ϕ' is 44.4^{o}. For σ_3 upto 100 kPa, c' is found to be zero for both types of reinforcement. With increased percentage of mesh elements or number of circular disc, in ϕ, upto

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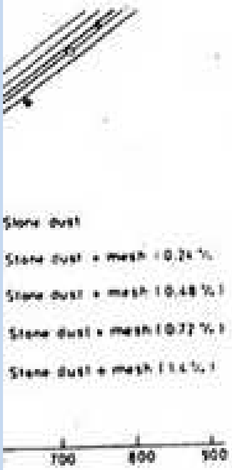
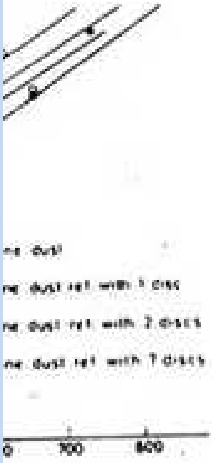


reinforced



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Stone Dust
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Dust % (wt) 0.72	1.4
0	0
51.4	54.0
36.4	65.0
46.6	46.0

Table 3: Strength Parameters for Stone Dust Reinforced with Geogrid Discs.

σ_3 kPa	Parameter	Stone dust with discs, (Number) 0	1	2	3
< 100	c' (kPa)	0	0	0	0
	ϕ' (deg)	44.4	51.4	57.0	60.0
> 100	c' (kPa)	0	9.6	115.0	146.0
	ϕ' (deg)	44.4	38.7	38.7	40.5

footing of width (B) at the depth (D) below ground level resting on stone dust of thickness (H) which in turn rests on a clay layer as shown in Fig.7 has been considered. For the case of micro-mesh reinforced stone dust the proportion of micro-mesh has been assumed to be distributed uniformly in the stone dust. One layer of geogrid has been assumed to be placed at the interface of stone dust and clay.

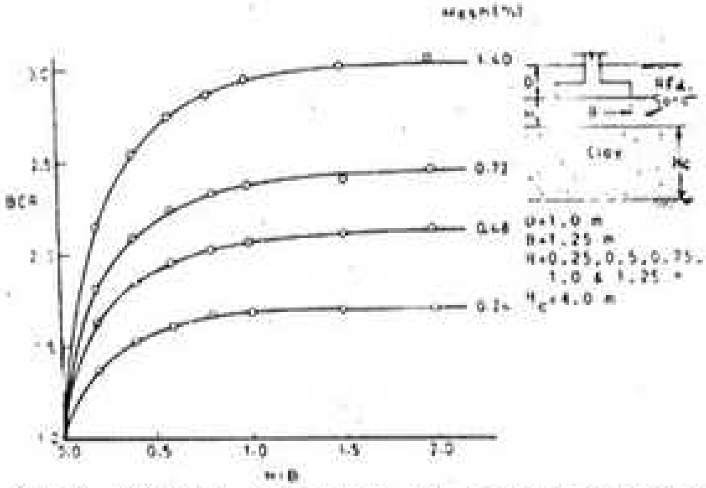

Fig.7 Typical variation of bearing capacity ratio with H/B for footing on soft clay overlay with micro-mesh reinforced stone dust

The computation of the ultimate bearing capacity of both reinforced as well as unreinforced stone dust has been carried out by using the approach for two layer system by Meyerhof (1974).

The BCR (ratio of ultimate bearing capacity with and without reinforcement) have been calculated and the variation in BCR with H/B for different percentages of micro-mesh is presented in Fig.7. In general it can be seen from this figure that the BCR increases with increase in H/B upto a value of around 1.5; beyond this, the increase is almost insignificant. The figure also shows an increase in BCR with increasing micro-mesh percent. Considering BCR of at least 1.5 as minimum expected improvement, all mesh percentages indicate this improvement at H/B values more than 0.4 upto a maximum of 3.0. The variation of BCR with H/B ratio for one layer of geogrid placed at the interface between stone dust and clay is illustrated in Fig.8. The figure shows