4th International Conference on

22

GROUND IMPROVEMENT TECHNIQUES (VOL. 2)

26 – 28 March 2002, Kuala Lumpur, Malaysia

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- Dept. of Civil Engineering, University of Malaya, Malaysia
- Portuguese Society for Geotechniques, Portugal
- International Association of Engineering Geology (IAEG) Commission No. 18: Collapsible soils / Commission de L'AIGI C:18: SOLs Effondrables
- School of Property & Construction, Nottingham Trent University, United Kingdom

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Geogrid micro-mesh reinforced granular trench

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Abstract

The results of conventional drained triaxial compression tests conducted on 100 mm diameter x 200 mm high specimens of sand reinforced with geogrids micro-meshes (GMM), mixed randomly with sand with various percentages up to 1.4 are presented in this paper. The effect of inclusion of GMM percentages on the strength of reinforced sand increases with increasing confining pressures, the strength increases gradually with increasing mesh percentages up to 0.72 beyond this value the strength tends to approach asymptotic, The strength parameters of the reinforced sand show significant improves. These experimental data are utilised to assess the overall influence of geogrids micro-meshes on the bearing capacity of granular trench problems. Granular trenches reinforced with geogrid micro-meshes installed in clay bed have been analyesed for bearing capacity and bearing capacity ratio. The sand as well as the reinforced sand with GMM distributed randomly is found improves the bearing capacity of granular trench.

Keywords: Geogrid, Reinforcement, Granular trench, Bearing capacity, BCR .

1. Introduction

The technique of soil reinforcement is being extensively used, since the last century, in a variety of applications ranging from earth retaining structures to subgrade stabilization. It is one of the most successful and reliable techniques used and it is fast replacing the other conventional improvement methods.

Recent research by different workers [1-5] on randomly distributed as well as oriented layer reinforcements is very much encouraging. In view of the above, triaxial tests have been conducted to understand the strength behavior of geogrid micro-meshes reinforced sand. The results have been used to assess the influence of reinforced sand on the bearing capacity of granular trench problems.

Granular trench: bearing capacity of weak clays can be improved by using granular pile or stone column. The two-dimensional plane-strain version of a granular pile is a granular trench. Madhav and Vitkar[6] investigated the latter problem and derived analytical expressions for various combinations of parameters. The ultimate bearing capacity (q_u) of the footing of the granular trench is determined by

$$q_u = C_2 Nc + (\gamma_2 B/2) N\gamma + Df\, \gamma_2 N_q \qquad (1)$$

Where,

Nc, $N\gamma$ and Nq are the dimensionless factors depend on the properties of trench and soil materials.

C_1, γ_1 and C_2, γ_2 are cohesion and unit weight of trench material and clay soil respectively, and Df is depth of foundation .

Bearing capacity ratio : a term bearing capacity ratio[7] (BCR) has been defined to compare the test data as :

$$BCR = q_{u(R)}/q_u \qquad (2)$$

or

$$q_{u(R)} = q_u + \Delta q_u \qquad (3)$$

$$q_{u(R)}/ q_u = 1 + \Delta q_u/ q_u$$

$$BCR = I + \Delta BCR \qquad (4)$$

Where, $q_{u(R)}$ = ultimate bearing capacity of the reinforced soil ,
q_u = ultimate bearing capacity of the unreinforced soil ,
Δq_u = change in the ultimate bearing capacity due to reinforcement inclusion and
ΔBCR = change in bearing capacity ratio .

In the case of strengthening the clay bed by granular trench of sand or reinforced sand with geogrid micro-meshes the BCR for both is:

$$BCR = 1 + \Delta BCR_{(s)} + \Delta BCR_{(R)} \quad (5)$$

$$\Delta BCR_{(s)} = \frac{q_{u(s)} - q_{u(c)}}{q_{u(c)}} \quad (6)$$

$$\Delta BCR_{(R)} = \frac{q_{u(R)} - q_{u(s)}}{q_{u(c)}} \quad (7)$$

Where, $\Delta BCR_{(s)}$=change in BCR brought out by unreinforced sand of granular trench ,
$\Delta BCR_{(R)}$=change in BCR due to contribution of reinforcement in the granular trench,
$q_{u(s)}$=ultimate bearing capacity of granular trench of sand,
$q_{u(c)}$ = ultimate bearing capacity of clay bed and
$q_{u(R)}$ =ultimate bearing capacity of granular trench of reinforced sand.

2. Experimental Programme

2.1 Soil

The investigation was carried out on a granular soil, comprising of sub angular particles. The relative density, uniformity coefficient and coefficient of curvature are 0.86, 3.35 and 0.84 respectively.

2.2 Reinforcement

The geogrid used was the extruded, unoriented variety made of high density polyethylene (HDPE), manufactured by Netlon India. Geogrids have been used as reinforcements in the form of micro-meshes designated as GMM. The micro-meshes are small square pieces of size 50x50mm cut out of geogrids. Many possible mesh element sizes were subjected to some exploratory tests and from these 50mm square elements were chosen as suitable. The physical and mechanical properties of geogrids are given in Table1.

Table 1 Properties of Geogrid Reinforcement

Type	CE 121
Polymer	Polyethylene
Aperture size	8 x 6 mm
Mass/unit area	730 g/m^2
Tensile strength	7.68 kN/m
Max. extension at max. load	20.2 %
Load at 10% extension	6.8 kN/m
Elongation at peak strength	3.2 %

2.3 Triaxial Tests

Saturated sand alone and sand mixed with various percentages up to 1.4 (by weight) of mesh element were deposited in layers in water into split mould forms. The specimens (100mm diameter and 200mm high) were prepared in a manner similar to that for specimens of saturated cohesionless soil for conventional consolidated drained triaxial test[8]. Each layer was compacted to achieve required density by vibration at a constant frequency. The achieved dry density of sand and sand-mesh mixtures varied from 17.6 to 18.0 kN/m^3. The cell pressures applied were 25, 50, 100 and 200 kPa at a deformation rate of 0.2 mm/minute .

3. Results

3.1 Stress – Strain

Typical stress-strain curves for unreinforced and reinforced sand with various percentages of GMM elements are presented in Figs. 1 and 2, under confining pressures of 25, 50, 100 and 200 kPa. In general, it is observed that the inclusion of GMM reinforcement increased the deviator stress developed at any strain level which confirms the ability of mash element to strengthen the sand. It can be found that the residual strength beyond the peak improves better than the maximum. It is seen that the peak stresses in reinforced sand occurred at slightly higher axial strain than the sand alone at lower confining pressure. The deviator stress seems to be linearly improving with micro-mesh content.

Figures also depict the volume change behaviour. These figures in general, reveal that both reinforced as well as unreinforced sands exhibit similar trends i.e an initial compression and then dilation as the axial strain increases. In general, it my be inferred that the effect of reinforcement is to decrease the volumetric expansion.

3.2 Effect of Inclusion

The effects of inclusion of GMM percentages on the strength of reinforced sand are illustrated in Fig.3, where the relation between major principal stress at failure (σ_{1f}) and GMM percentage is presented. The figure shows that as the mesh percentage increases, the strength increases with increasing confining pressures. The increase in strength is generally inproportion to the amount of reinforcement i.e GMM percentages. It is observed that in the reinforced sand the strength increases gradually with increasing mesh percentages upto 0.72 % beyond this value the strength tends to approach an asymptotic.

3.3 Strength Ratio

The values of strength ratio (the ratio of the strength of a reinforced sand to that of an unreinforced sand under different confining pressures) are given in Table 2. This table in general, indicates an increase in strength ratio with mesh percentages and also a general decrease in strength ratio with- increase in confining pressure .

Table 2 Strength Ratio for GMM Reinforced Sand

σ_3 (kPa)	GMM Reinforcement (%)			
	0.24	0.48	0.72	1.4
25	1,25	1.45	1.69	2.19
50	1.07	1.12	1.18	1.72
100	1.07	1.13	1.18	1.50
200	1.13	1.21	1.39	1.42

3.4 Strength Characteristics

The variation between σ_{1f} and corresponding σ_3 for unreinforced as well as reinforced sand with various percentages of GMM are shown in Fig.4 . It is interesting to note from this figure that the failure envelope is linear for unreinforced sand whereas, it becomes bilinear with reinforcements. The critical confining pressure (σ_c) is found to be around 25 kPa . The equivalent confining stress increase ($\Delta\sigma_c$) due to micro-mesh reinforcement was determined from the observed increase in major principal stress at failure ($\Delta\sigma_{1f}$) according to the following relationship :

$$\Delta\sigma_3 = \sigma_3 \frac{(\Delta\sigma_{1f})}{\sigma_{1f}} \qquad (8)$$

3.5 Strength Parameters

The p-q plot for unreinforced sand and reinforced sand with various percentages of geogrid micro-meshes are presented in Fig.5, for different confining pressures. The failure envelops are observed to be bilinear. This figure indicates that the range of confining pressure upto 50kPa corresponds to initial linear portion, whereas the second linear portion corresponds to the range 50-200 kPa. The values C´ and φ' obtained are presented in Table3. A study of this table reveals the following:

- For unreinforced sand the values of C´ is zero upto σ_3 = 200 kPa.
- For the range of confining pressure upto 50 kPa, sand when reinforced with mesh percent upto 1.4 %, the value of φ' increases from 44.4° to 54° .
- For the range of σ_3 from 50 to 200 kPa, sand when reinforced with mesh percent upto 1.4 %, C´ increases from zero to 65 kPa .
- In general, φ' nearly remains constant at the same value of unreinforced sand.

The ratios of reinforced friction angle φ_R[9] were calculated and summarized in Table 3. The results appeared to indicate that the friction angle ratio was greatly affected by the increases of mesh percentages.

Based on the preceded observations it may be stated that the pseudo-cohesion of reinforced sand with bond failure, and failure by lack of adhesion and slipping of the meshes were characterized by an apparent friction angle (φ_R) which is larger than that of the sand alone, and both (φ_R) and the critical confining pressure (σ_c) were mainly influenced by the soil-inclusion, interface friction and the inclusion percentages. As shown in Fig.4 the ultimate strength of reinforced sand failed by inclusion was governed primarily by the mesh percentages, with the more extensible inclusion (GMM elements) the improvement

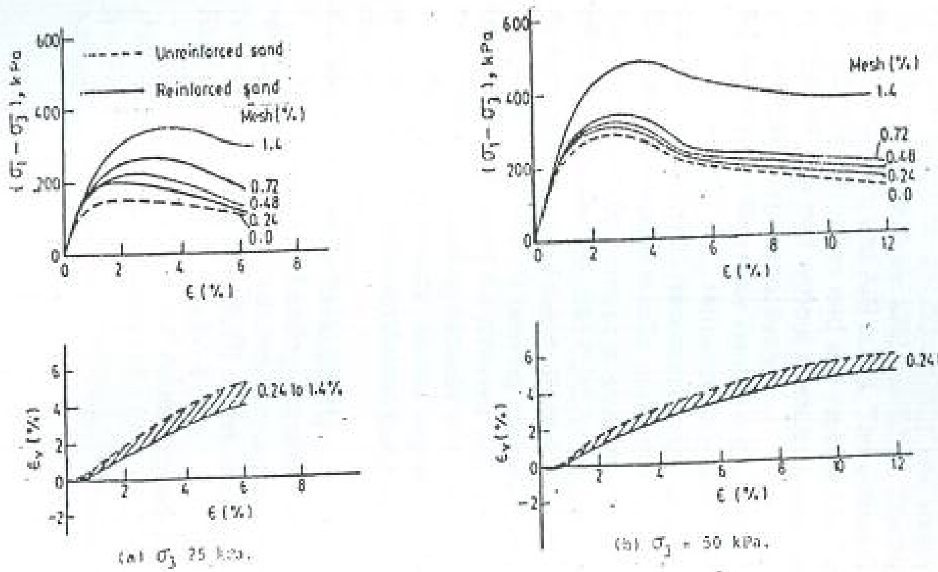

FIG. 1 STRESS - STRAIN - VOLUME CHANGE RELATIONSHIP FOR SAND REINFORCED WITH GMM AT DIFFERENT σ_3 FOR 25 & 50kPa

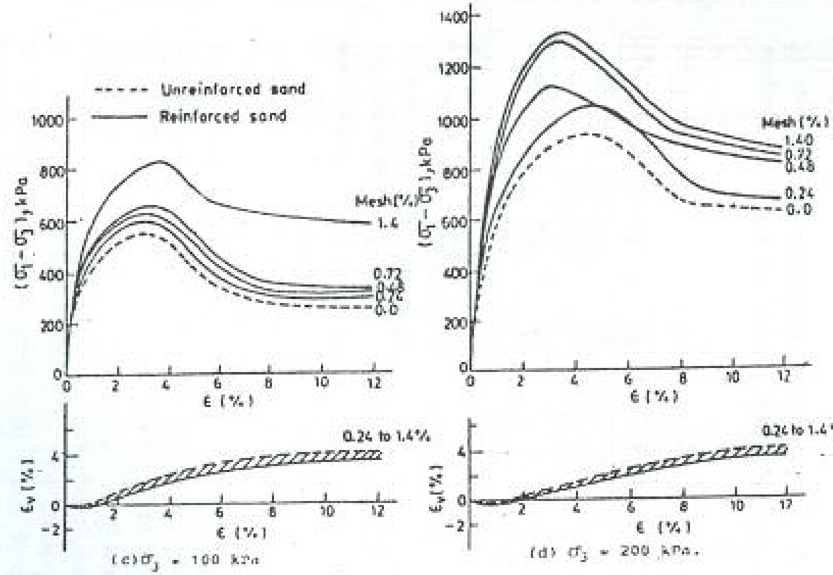

FIG. 2 STRESS - STRAIN - VOLUME CHANGE RELATIONSHIP FOR SAND REINFORCED WITH GMM AT DIFFERENT σ_3 FOR 100 & 200kPa

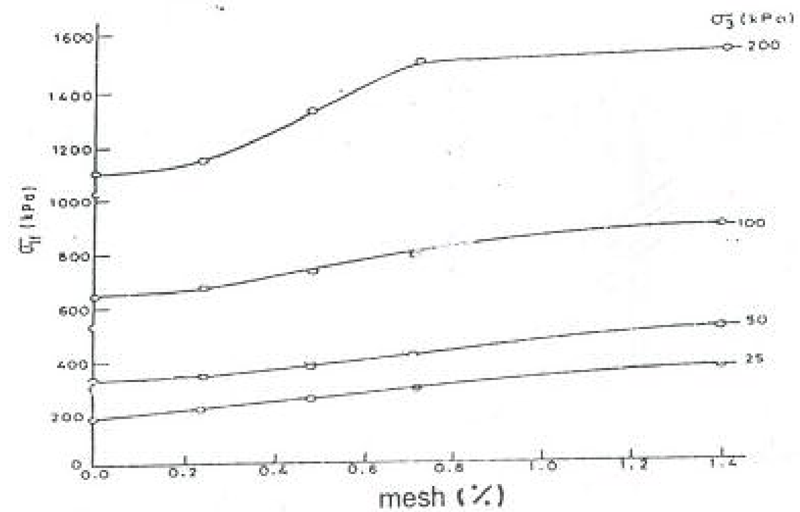

FIG.3 VARIATION OF σ_3 WITH MESH PERCENTAGES

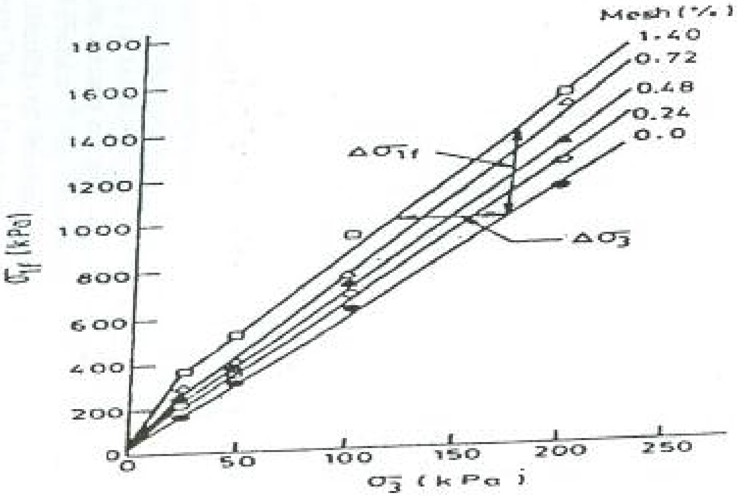

FIG. 4 VARIATON OF σ_{1f} WITH σ_3 REINFORCED SAND WITH GMM

in strength of reinforced sand was significant because the interaction with the soil particles through surface friction as well as the interlocking mechanism.

Table 3 Strength Parameters for Sand Reinforced with GMM

σ_3 (kPa)	Para-meter	Mesh, %								
		0	0.24	φ_R/φ'	0.48	φ_R/φ'	0.72	φ_R/φ'	1.4	φ_R/φ'
50	C'(kPa)	0	0		0		0		0	
	φ'(deg)	44.4	φ_R=48.8	1.1	50.1	1.13	51.4	1.16	54.0	1.22
50-200	C_R(kPa)	0	21		36		36		65	
	φ'(deg)	44.4	φ_R=45.5	1.02	45.5	1.02	46.6	1.05	46.0	1.04

4.Granular Trenches

The use of granular trenches is one of the recent techniques for improving the load carrying capacity of soft soils. The arrangement of such a granular trench installed in a surrounding weak clay deposit is shown in Fig. 6, a and b • Madhav and Vitkar[6] analysed the problem of a granular trench and derived an expression for the ultimate bearing capacity of footing on granular trench in soft soil as given by Eqn.1.

Herein, an analysis has been carried out to understand the changes brought out in ultimate bearing capacity of such a footing on granular trench when the reinforcements are introduced into the trench material (sand, in this case). The analysis is done for geogrid micro-meshes (GMM) reinforced sand used for granular trench as illustrated in Fig.6, c • For this, the weak clay deposit has been assumed to possess cohesion (C_2) of 20 kPa. Whereas, the values of cohesion C_R of reinforced material for granular trench (C_1 is replaced by C_R for reinforced material) adopted herein are based on the pseudo-cohesion concept suggested by Schlosser and Long[10] and used by Gray and Al-Refiea[11] for similar analysis. In this study the values of C_R = 36 kPa have been extracted from the results of triaxial tests conducted on the corresponding material (Table 3). The footing is placed at a depth Df = 1.0 m below ground level and rests directly on granular trench(Fig.6) . The footing widths (B) varied are 1.0,1.5 and 2.0 m • The granular trench width (A) is to varied as to obtain A/B ratios from 0.8 to 2 in steps of 0.2 .

4.1 Unreinforced Granular Trench

The value of C_2 remaining constant (20 kPa), Nc values depend on C_1 which equal zero in this case sand . $N\gamma$ based on the ratio of the unit weight of the granular trench γ_1 and γ_2 of the clay are 17.8 and 15.7 kN/m^3 respectively which is 1.13 . The Nq based on the internal friction angle of the sand (granular trench φ= 44.4°), and A/B ratios. The ultimate bearing capacity values have been computed from Eqn.(1) for different A/B ratios and are reported in Table 4. The term $q_{u(c)}$ in Table 4 denotes ultimate bearing capacity of clay bed without granular trench and $q_{u(s)}$ for ultimate bearing capacity of granular trench for unreinforced sand respectively .

4.2 Reinforced Granular Trench with GMM

The values of C_2 remaining constant (20 kPa), Nc values should depend on C_R. Thus, to derive higher Nc, it becomes necessary to select appropriate mesh percentage so as to achieve higher possible C_R The experimental results presented in Fig. 3 indicate GMM percentage of 0.72 is optimum to derive maximum possible increase in strength. The corresponding C_R value extracted from Table3 (under σ_3>50 kPa) is 36 kPa and the ratio C_R/C_2 in this case worked out to be 1.8 • The friction angles of reinforced sand φ_R from the triaxial test results is 46.6°(Table3). Adopting these values of C_R/C_2 and φ_R and using Madhav and Vitkar's[6] charts of bearing capacity factors and equation (1), the ultimate bearing capacity values have been computed for different A/B ratios and are presented in Table 4 . The term $q_{u(R)}$ in Table 4 denotes ultimate bearing capacity of reinforced granular trench with GMM.

The table distinctly shows significant improvement in ultimate bearing capacity due to GMM reinforced sand over unreinforced sand as a granular trench material for all the combinations of B and A/B ratio analysed here.

4.3 Bearing Capacity Ratio (BCR)

The changes in bearing capacity ratios of $\Delta BCR_{(s)}$ and $\Delta BCR_{(R)}$ due to the unreinforced and reinforced granular trench are computed by using Eqns.6 and 7 respectively and given in Table 4, the bearing capacity ratios (BCR) of the unreinforced and reinforced granular trench are computed by using Eqn 5, are presented in Table 4 .

To understand the individual contribution brought out by sand alone and reinforcement towards changes in bearing capacity ratio (ΔBCR), the values of BCR are plotted against A/B for different values of B in Fig.7. This figure, in general indicate a bilinear increase in both $\Delta BCR_{(s)}$ and $\Delta BCR_{(R)}$ with increase in A/B ratio as well as B .

Table 4 : Ultimate Bearing Capacity and BCR of Granular Trench with and without GMM Reinforcements Sand

A	A/B	$q_{u(c)}$ kN/m^2	$q_{u(s)}$ kN/m^2	$q_{u(R)}$ kN/m^2	$\Delta(BCR)_s$	$\Delta(BCR)_R$	BCR	
							Unreinforcements Sand	Reinforcements Sand
B= 1.0m								
0.8	0.8	118	371	487	2.14	0.98	3.14	4.12
1.0	1.0	118	517	735	3.38	1.85	4.38	6.23
1.2	1.2	118	567	830	3.81	2.23	4.81	7.04
1.4	1.4	118	607	904	4.14	2.52	5.14	7.66
1.6	1.6	118	643	990	4.45	2.94	5.45	8.39
1.8	1.8	118	684	1077	4.80	3.33	5.80	9.13
2.0	2.0	118	738	1151	5.25	3.50	6.25	9.75
B= 1.5m								
1.2	0.8	118	396	516	2.36	1.02	3.36	4.38
1.5	1.0	118	553	783	3.96	1.95	4.69	6.64
1.8	1.2	118	616	889	4.22	2.31	5.22	7.53
2.1	1.4	118	666	974	4.64	2.61	5.64	8.25
2.4	1.6	118	710	1073	5.02	3.08	6.02	9.10
2.7	1.8	118	761	1171	5.45	3.48	6.45	9.93
3.0	2.0	118	826	1257	6.00	3.65	7.00	10.65
B= 2.0m								
1.6	0.8	118	422	545	2.58	1.04	3.58	4.62
2.0	1.0	118	590	830	4.00	2.03	5.00	7.03
2.4	1.2	118	666	948	4.64	2.39	5.64	8.03
2.8	1.4	118	725	1044	5.14	2.70	6.14	8.84
3.2	1.6	118	779	1155	5.60	3.19	6.60	9.79
3.6	1.8	118	839	1264	6.11	3.60	7.11	10.71
4.0	2.0	118	915	1363	6.75	3.80	7.75	11.55

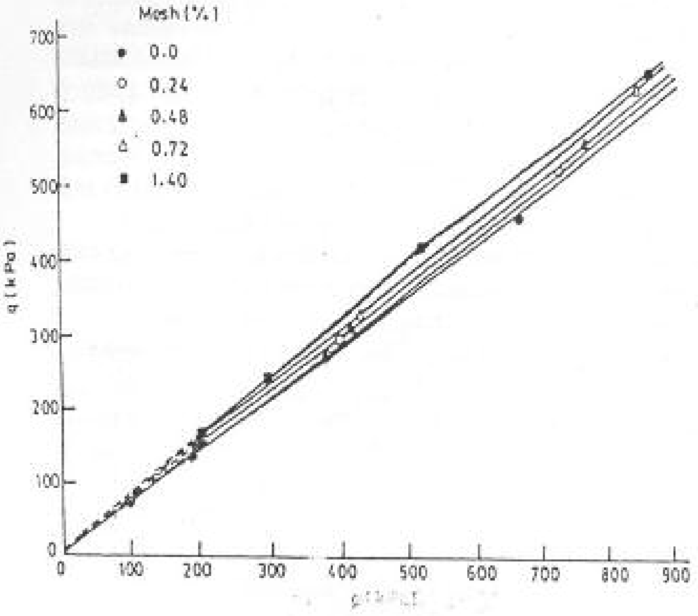

FIG . 5 p-q PLOTS FOR SAND REINFORCED WITH GMM.

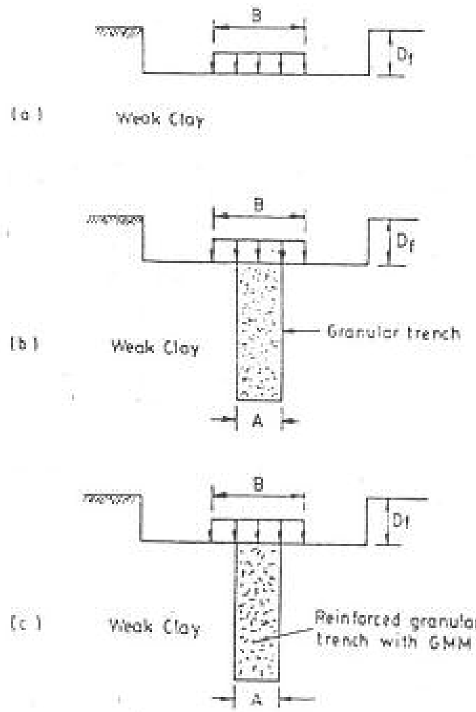

FIG. 6 GRANULAR TRENCH WITH VARIOUS TYPES OF REINFORCEMENT.

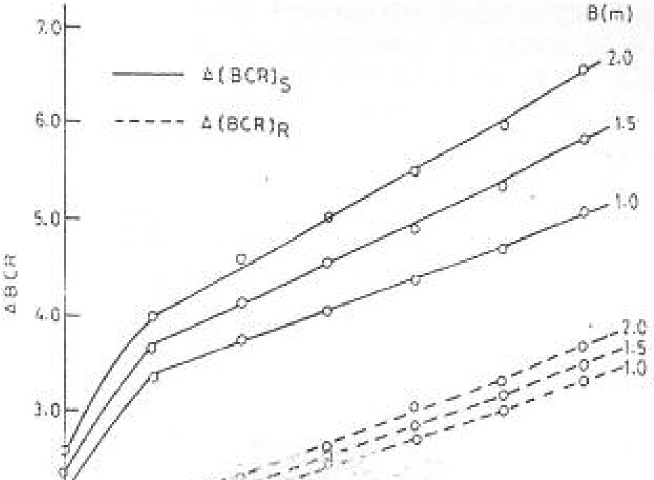

FIG.7 VARIATION OF Δ BCR WITH A\B FOR UNREINFORCED AND GMM REINFORCED SAND GRANULAR TRENCH.

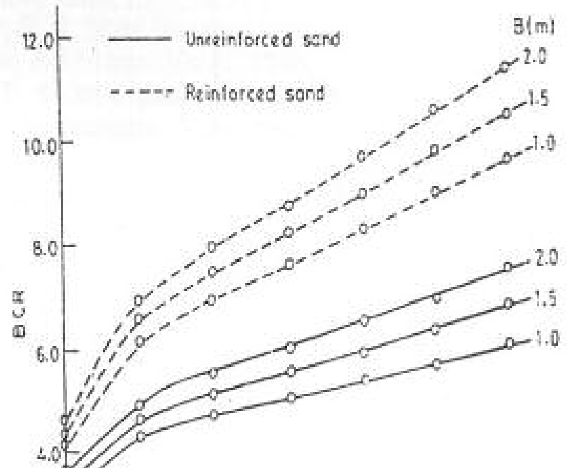

FIG.8 VARIATION OF BCR WITH A\B WITH AND WITHOUT GMM REINFORCMENT SAND

The variations of BCR with A/B ratio of the granular trench with unreinforced and reinforced sand are shown in Fig. 8 . This figure clearly exhibits a bilinear increase in BCR with increase in A/B ratio for both unrinforced as well as reinforced granular trenches. For any particular value of A/B and footing width (B), the values of reinforced trench are significantly higher than those for unreinforced trench indicating a distinct improvement due to inclusion of reinforcements. For example, granular trench with A/B of 1.2 and footing width 1.5m (Fig.8), the BCR values are 7.50 and 5.25 with GMM reinforced sand and unreinforced sand respectively indicating a BCR improvement of the order 2.25 over and above that of unreinforcod sand. Similar results have been observed for other cases. It is interesting to note from Figs. 7 and 8 that the intersection points of linear segments for all these cases invariably appears at A/B of 1.0

The problems of footing and granular trenches in clay beds analysed here on the basis of experimental results distinctly show the effectiveness and applicability of reinforcement to improve the ultimate bearing capacity and subsequently bearing capacity ratios.

It may also be noted that the analysis carried out in the present study is only indicative of the possible improvements as the actual improvement depends on the choice of correct reinforced soil parameters and the dimensions and depth of foundation/trench. Using these appropriate parameters determined in the laboratory can only reflect the magnitude of the improvement in BCR or settlement reduction

5. Conclusion

1. The geogrid micro-mesh increased the deviator stress developed at any strain including peak and residual levels
2. The peak stresses in the soil-mesh mixture occurred at slightly higher axial strain than the sand alone at lower cell pressure.
3. The critical confining pressure defines two distinct zones for the failure envelopes, (a) $\sigma_c > \sigma_3$ depicting an increase in anisotropic cohesion with nearly constant angle of internal friction and (b) $\sigma_c < \sigma_3$ depicting an increase of the internal friction angle only.
4. Without the inclusion of the reinforcement the ultimate bearing capacity of granular trench increases with A/B ratios.
5. With the inclusion of the reinforcements the ultimate bearing capacity of granular trench increases further.
6. Unreinforced as well as reinforced granular trenches exhibit a bilinear increase in BCR with increase in A/B ratio

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