SOIL IMPROVEMENT WITH GEOSYNTHETICS

AMELIORATION DU SOL AU MOYEN DE MATERIAUX GEOSYNTHETIQUES

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SYNOPSIS: The results of conventional drained triaxial compression tests conducted on 100mm diameter x 200mm high specimens of two types of sands reinforced with woven and nonwoven geotextilesand geogrids, in the form of circular discs as well as micro-meshes are presented in this paper. This experimental data is utilised to assess the overall influence of such reinforced material overlying clay beds on the bearing capacity and settlement of footings as well as granular trench problems. The reinforcements are found to improve only the bearing capacity. The influence on settlement is marginal.

INTRODUCTION

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

Recent research by different workers (Mercer et al. 1984; McGown et al 1985) 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 behaviour of micro-mesh as well as oriented layer(s) reinforced sands. The results have been used to assess the influence of such reinforced material overlying clay beds on the bearing capacity and the settlement of footings as well as granular trench problems.

EXPERIMENTAL WORK

Materials

The investigation was carried out on two granular materials viz., fine grained micacious Yamuna sand (S1) and crushed stone dust (S2) comprising of subangular particles. The relative density, uniformity coefficient and coefficient of curvature for Yamuna sand are 0.60, 1.76 and 1.09 respectively whereas for stone dust these are 0.86, 3.35 and 0.84.

Three geosynthetics GTW, GTNP and GG (properties included in Table-1) were used as oriented circular disc reinforcements. The geogrid (GG) cut into pieces of sizes 30x30 and 50x50 mm was used as micro-mesh (GMM) reinforcement.

Triaxial tests

The specimens (100 mm diameter and 200 mm high) were prepared in a manner similar to that for specimens of saturated cohesionless soil for conventional consolidated drained triaxial tests, Bishop & Henkel (1962). The reinforcement disc was placed on the already compacted (by vibration) and levelled granular material layer of predetermined height. The procedure was repeated till the full height of the specimen was reached by building up layer by layer. In case of GMM reinforcement, a predetermined percentage of micro meshes were mixed thoroughly with granular material and such mixture was placed, at densities ensuring uniformity throughout.

Parameters Varied

The compaction density of Yamuna sand was maintained at 15 ± 0.2 kN/m^3 whereas, stone dust was compacted at 17.8 ± 0.2 kN/m^3. The number of reinforcement discs was varied from 1 to 7 and accordingly the ratios of specimen radius (r) to reinforcement spacing (ΔH) were in the range of 0.5 to 2.0. For GMM, percentages studied were upto 1.4. The cell pressures applied were 25,50,100,200 and 400 kPa. The specimens, after consolidation were sheared in drained condition at a deformation rate of 0.2 mm/minute.

Results

The summary of the typical triaxial test results is presented in Figs.1 and 2 for geotextiles and in Tables 2 to 4 for geogrids.

Table 1: Properties of Geosynthetics

Designation	Structure	Polymer	Aperture size (mm)	Mean pore size (micron)	Thick-ness (mm)	Mass per unit area (g/m^2)	Tensile strength (kN/m)	Extension at failure (%)	Secant modulus @ 10% elongation (kN/m)
GTW	Woven	Polypro-pylene	-	25	0.70	270	*MD 37.00 **MD 33.90	MD 28.0 CD 26.0	170.0
GTNP	Non-woven	Polypro-pylene	-	75	4.02	275	MD 14.41 CD 14.03	MD 56.6 CD 66.5	1.1
GG	Grid	High Density Polyethy-lene	8x6	-	3.30	730	MD 7.80 CD 6.50	MD 34.0 CD 43.0	60.0

* MD - Machine direction ** CD - Cross direction

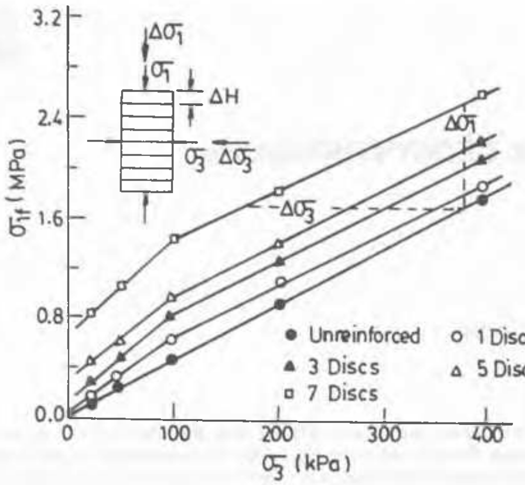

Fig.1 Variation between σ_{1f} and σ_3 for S1 Reinforced with GTW.

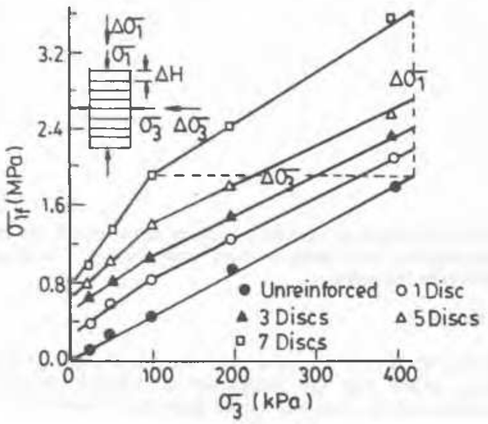

Fig.2 Variation between σ_{1f} and σ_3 for S1 Reinforced with GTNP.

Table 2: Strength Parameters for Geogrid Disc Reinforced Sand S2 ($\gamma = 17.8$ kN/m^3)

σ_3 (kPa)	Parameter	No. of reinforcement discs 0	1	2	7
< 100	ϕ'(deg.)	44.4	51.4	57.0	60.0
	c'(kPa)	0	0	0	0
> 100	ϕ'(deg.)	44.4	38.7	38.7	40.5
	c'(kPa)	0	9.6	115.0	146.0

Table 3: Strength Parameters for GMM Reinforced Sand S1

σ_3	Parameter	Loose sand ($\gamma = 14.0$ kN/m^3)		Dense sand ($\gamma = 16.0$ kN/m^3)			
(kPa)		Mesh %		Mesh %			
		0	1.40	0	0.24	0.72	1.40
<50	c'(kPa)	0	0	0	0	0	0
	ϕ'(deg)	36.9	44.0	40.6	44.0	46.5	48.0
50-200	c'_R(kPa)	0	43.8	0	25	33	57
	ϕ'(deg)	36.0	37.0	40.6	38.5	39.0	39.0

Table 4: Strength Parameters for GMM Reinforced Sand (S2)

σ_3 (kPa)	Parameter	Mesh, % 0	0.24	0.48	0.72	1.40
< 50	c'(kPa)	0	0	0	0	0
	ϕ'(deg)	44.4	48.8	50.1	51.4	54.0
50-200	c'_R(kPa)	0	21	36	36	65
	ϕ'(deg)	44.4	45.5	45.5	46.6	46.0

APPLICATION TO FOOTINGS

Bearing Capacity Ratio

For the analysis, a square footing (width, B) placed at depth, D_f below ground level and resting on sand layer overlying a clay bed has been considered (Fig.3). The values of B and D_f adopted were 1.25 m and 1.00 m respectively and the clay as having unit weight (γ_t) of 15.70 kN/m^3, undrained cohesion (c) of 19 kPa. In all, six cases have been analysed.

The ultimate bearing capacity (UBC) has been computed using Meyerhof's theory (1974). The UBC of reinforced sand (q_{uR}) can be expressed as

$$q_{uR} = q_{uS} + \Delta q_u$$

where, q_{us} = UBC of sand, and
Δq_u = Change in UBC due to reinforced inclusion.

Hence, $q_{uR}/q_{us} = 1 + \Delta q_u/q_{us}$

or bearing capacity ratio, BCR = 1+ΔBCR

where, ΔBCR = Change in BCR.

Using the values of UBC the bearing capacity ratios have been calculated for various H/B ratios (where H = thickness of granular layer, below the foundation) and presented in Fig.3 for GMM reinforced sand (S2). In general, it can be seen that the BCR increases with increase in H/B upto a value of around 1.5, beyond this, the increase is insignificant. The figure further shows an increase in BCR with increasing mesh percentage which is as expected. If one considers BCR of at least 1.5 as the

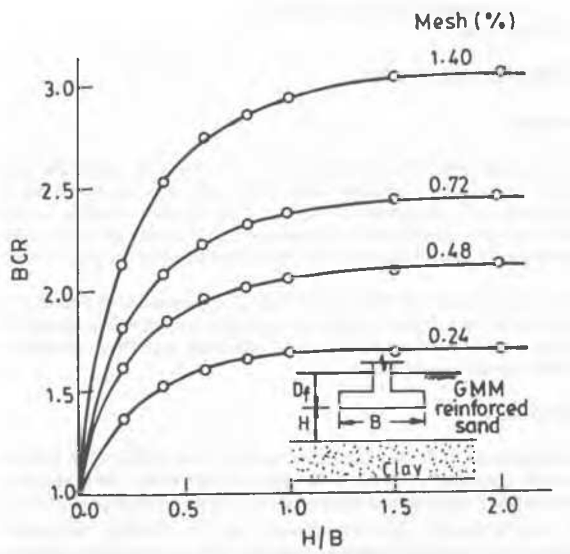

Fig.3 Variation of BCR with H/B for GMM Reinforced sand S2 Overlying Clay.

minimum expected improvement, this is achieved at a low mesh percentage of 0.24 at H/B of only 0.4. The maximum BCR obtained in this study is over 3.

Similar conclusions may be drawn from Fig.4 which presents results for GMM reinforced sand (S1) in loose and dense conditions. Also higher BCR values are observed for sand in loose condition.

The results for single layer of geosynthetic placed at sand-clay interface are illustrated in Fig.5 for geogrid and sand (S2)-clay interface and in Fig.6 geotextile at sand-clay interface. These figures show curvilinear increase in BCR with H/B upto H/B of 1.5, beyond which the increase is insignificant.

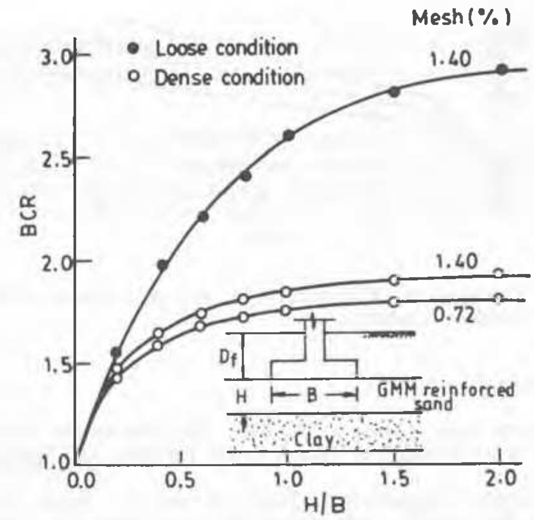

Fig.4 Variation of BCR with H/B for Footing on GMM Reinforced Sand S1 Overlying Clay.

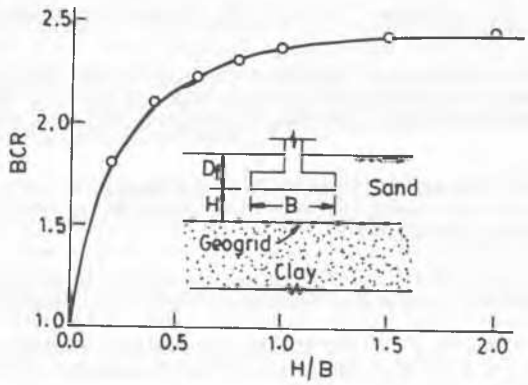

Fig.5 Variation of BCR with H/B for Reinforced Sand S2 with GG Layer at Interface.

Settlement

The consolidation settlement for different cases has been estimated by dividing the entire clay bed of thickness H_c into six sub-layers each of thickness 0.5 B, and the effective stresses required for settlement computations have been worked out accordingly at the centre of each such sub-layer. For the case of footing directly resting on clay layer, the pressure increment at the centre of each sub-layer due to the loaded footing has been computed using Boussinesq's pressure isobars. Other cases of two layered system wherein footing rests in sand (unreinforced/reinforced) overlying clay bed the method suggested by Fox (1948) has been used to compute the required axial interface stresses. The consolidation settlement(s) for different cases have been estimated from the consolidation settlement equation assuming the values of compression index and initial void ratio as 0.35 and 1.1 respectively.

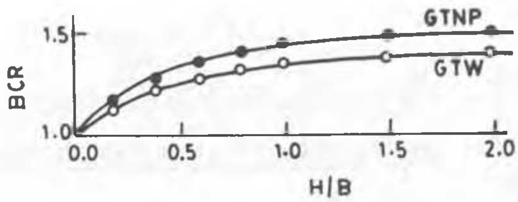

Fig.6 BCR variation with H/B for Sand S1 Reinforced with Single Layer of Geotextile at Interface.

The influence of GMM reinforcement on the consolidation settlement is illustrated in Fig.7 which shows the variation of settlement with H/B ratio for different mesh percentages in sand S2. It is evident that the reinforcement does reduce the settlement, but the reduction in settlement is marginal in comparison with unreinforced sand. Similar trends have been noticed for the case of GMM reinforced sand S1 both in loose and dense state.

In case of sand S2 reinforced with layer of geogrid GG placed at interface of sand and clay, the settlement decreases with increase in H/B ratio as illustrated in Fig.8. The reduction in settlement due to introduction of geogrid layer is again marginal.

Though the computations have been made for particular cases, it may generally be inferred that geogrid reinforcement (in the form of GMM or layers of GG) contributes significantly to BCR improvement but is hardly influential in reducing the settlement. These observations more or less substantiate the earlier work on geogrid layer reported by Yamanouchi (1972) and Yasudharee et al. (1986).

Reinforced granular trench

An analysis has been carried out to understand the changes brought out in ultimate bearing capacity of a footing on granular trench (Fig.9) when the reinforcements are introduced into the trench materials following the procedure developed by Madhav and Vitkar (1978).

For this, the weak clay deposit has been assumed to possess cohesion (C_2) of 20 kPa. The values of cohesion c_R of reinforced material for granular trench (C_1 is replaced by c_R for reinforced material) adopted herein the based on the pseudo-cohesion concept suggested by Schlosser and Long (1974). In this study the values of c_R have been extracted from the results of triaxial tests conducted on the corresponding material. The values of equivalent confining stress ($\Delta\sigma_3$) and increase in major principal stress at failure ($\Delta\sigma_{1f}$) were obtained from Fig.1 for GTW fabric and from Fig.2 for GTNP fabric.

The footing was assumed to be placed at a depth (D_f) of 1.0m below ground level and rests directly on granular trench. The footing widths (B) varied are 1.0, 1.5 and 2.0m. The granular trench width (A) is so varied as to obtain A/B ratios from 0.8 to 2.0.

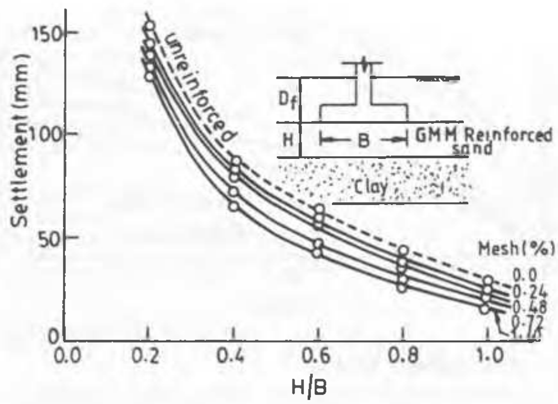

Fig.7 Variation of Settlement with H/B for Footing on GMM Reinforced Sand S2 Overlay Clay Layer.

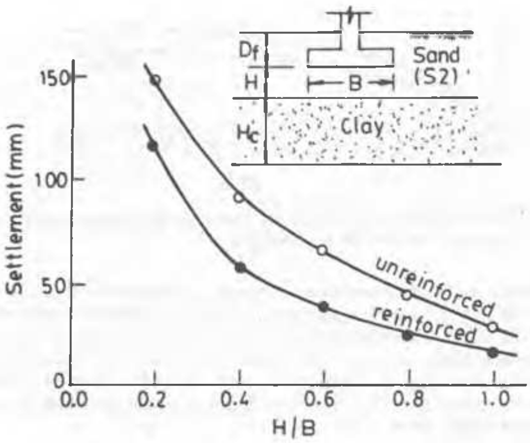

Fig.8 Variation of Settlement with H/B Ratio for Footting (GG Layer at Interface of Sand S2 and Clay).

The typical variations of BCR with A/B ratio for different values of B are illustrated in Figs.9&10 for sand S2 with GMM and sand S1 with GTNP fabric respectively. These figures clearly exhibit a bilinear increase in BCR with increase in A/B ratio for both unreinforced as well as reinforced granular trenches. For any particular value of A/B and footing width, the BCR values of reinforced trench are significantly higher than those for unreinforced trench indicating a distinct improvement due to the inclusion of reinforcements. Similar results have been observed for other types of reinforcements. It is interesting to note from these that the intersection points of linear segments for all these cases invariably appears at A/B of 1.0

CONCLUSIONS

The analysis indicates improvements in ultimate bearing capacity and consequently the bearing capacity ratios for all cases. The contribution of the reinforcements towards settlement reduction is only marginal.

It may also be noted that the analysis carried out herein 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.

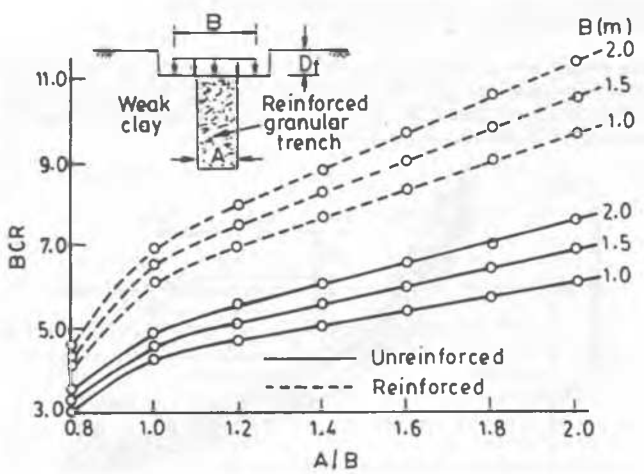

Fig.9 Variation of BCR with A/B with and without GMM Reinforcement Sand S2.

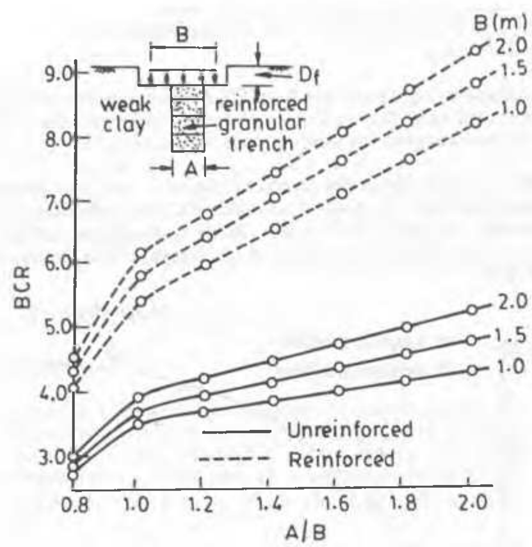

Fig.10 Variation of BCR with A/B with and without GTNP Reinforced Sand S1.

REFERENCES

Bishop, A.W. and Henkel, D.J. (1962). *The Measurement of Soil Properties in the Triaxial Test*, Edward Arnold Publishers Ltd., London.

Fox, L. (1948). Computation of Traffic Stresses in a Simple Road Structure, *Proc. 2nd Int. Conf. on Soil Mech. and Found. Engg.*, Vol.2, pp.236-246.

Madhav, M.R. and Vitkar, P.P. (1978). Strip Footing on Weak Clay Stabilized with a Granular Trench of Pile, *Canadian Geotechnical J.*, Vol.15, pp.605-609.

McGown, A., Andrawes, K.Z., Hytrisis, N, and Mercer, F.B. (1985). Soil Strengthening Using Randomly Distributed Mesh Element, *Proc. 11th Int. Conf. Soil Mech. and Found. Engg.*, San Francisco, Vol.III, pp.1735-1738.

Mercer, F.B., Andrawes, K.Z., McGown, A. and Hytrisis, N. (1984). A New Method of Soil Stabilization, *Proc. Symp. Polymer Grid Reinf. in Civil Engg.*, London, pp.244-249.

Meyerhof, G.G. (1974). Ultimate Bearing Capacity of Footing on Sand Layer Overlying Clay, *Canadian Geotechical J.*, Vol.11, No.2, pp.223-229.

Schlosser, F. and Long, N.T. (1974). Recent Results in Franch Research on Reinforced Earth, *Proc. ASCE, Journal of the Construction Division*, Vol.100, GT3, pp.233-237.

Yamanouchi, T. (1972). Experimental Study on the Improvement of the Bearing Capacity of Soft Ground by Laying Resinous Net, *Proc. Symp. Foundation on Interbedded Sand*, Perth, pp.102-108.

Yassudharee, K., Hirao, K. Mieura, N. Yamanouchi T., Ryokai, K. (1986). The Use of Geotextiles Against Settlement of Soft Clay Under Cyclic Loading, *Proc. 3rd Int. Conf. on Geotextiles*, Vienna, Vol.1, pp.193-198.