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# TREIZIEME CONGRES INTERNATIONAL DE MECANIQUE DES SOLS ET DES TRAVAUX DE FONDATIONS

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# 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 $15\pm0.2$ kN/m$^3$ whereas, stone dust was compacted at $17.8\pm0.2$ 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 ($\Delta$H) 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<br>**MD 33.90 | MD 28.0<br>CD 26.0 | 170.0 |
| GTNP | Non-woven | Polypro-pylene | - | 75 | 4.02 | 275 | MD 14.41<br>CD 14.03 | MD 56.6<br>CD 66.5 | 1.1 |
| GG | Grid | High Density Polyethy-lene | 8x6 | - | 3.30 | 730 | MD 7.80<br>CD 6.50 | MD 34.0<br>CD 43.0 | 60.0 |

* MD - Machine direction ** CD - Cross direction

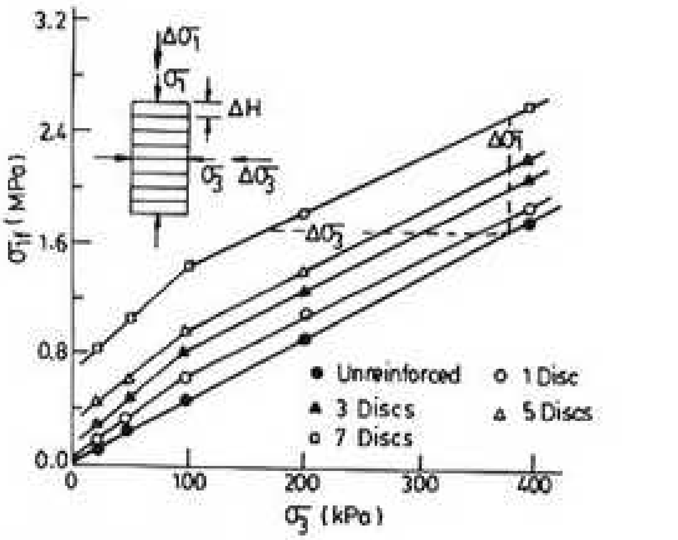


Fig.1 Variation between $\sigma_{1f}$ and $\sigma_3$ for S1 Reinforced with GTW.

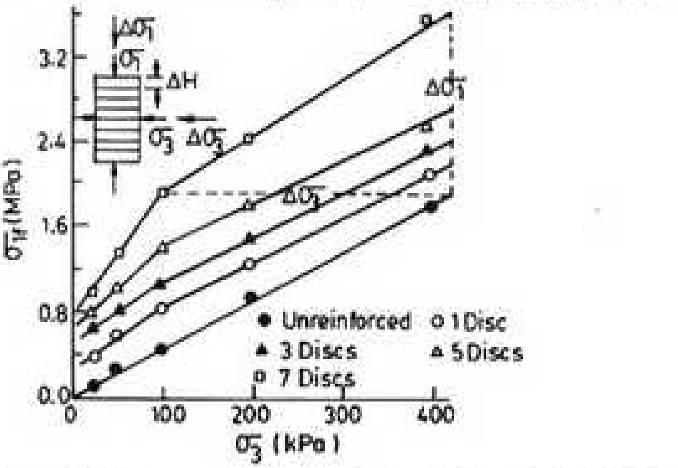


Fig.2 Variation between $\sigma_{1f}$ and $\sigma_3$ for S1 Reinforced with GTNP.

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

| $\sigma_3$ | Parameter | No. of reinforcement discs | | | |
|---|---|---|---|---|---|
| (kPa) | | 0 | 1 | 2 | 7 |
| < 100 | $\phi'$(deg.) | 44.4 | 51.4 | 57.0 | 60.0 |
| | c'(kPa) | 0 | 0 | 0 | 0 |
| > 100 | $\phi'$(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

| $\sigma_3$ | 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 |
| | $\phi'$(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 |
| | $\phi'$(deg) | 36.0 | 37.0 | 40.6 | 38.5 | 39.0 | 39.0 |

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

| $\sigma_3$ | Parameter | Mesh, % | | | | |
|---|---|---|---|---|---|---|
| (kPa) | | 0 | 0.24 | 0.48 | 0.72 | 1.40 |
| < 50 | c'(kPa) | 0 | 0 | 0 | 0 | 0 |
| | $\phi'$(deg) | 44.4 | 48.8 | 50.1 | 51.4 | 54.0 |
| 50-200 | $c'_R$(kPa) | 0 | 21 | 36 | 36 | 65 |
| | $\phi'$(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 ($\gamma_c$) 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
$\Delta q_u$ = Change in UBC due to reinforced inclusion.

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

or bearing capacity ratio, BCR = 1+$\Delta$BCR

where, $\Delta$BCR = 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

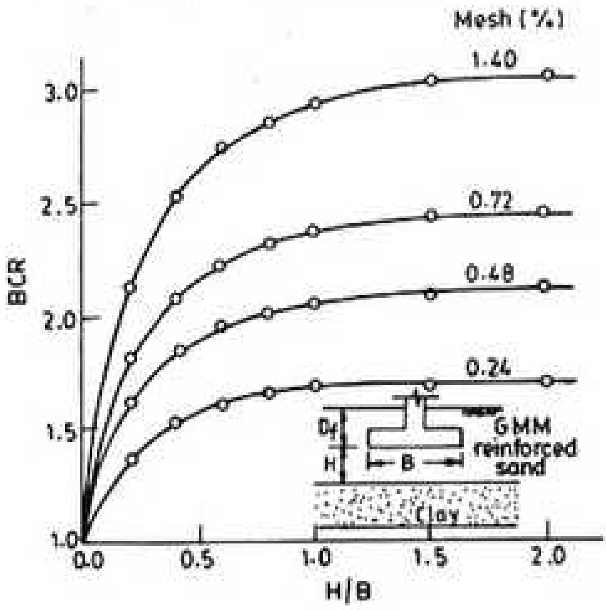


Fig.3 Variation of BCR with H/B for GMM Reinforced sand S2 Overlying Clay.

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