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Geo-reinforced sand columns: Small scale experimental tests and theoretical modelling

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ABSTRACT: The mechanical behaviour of sand columns reinforced by means of geosynthetics is studied by performing small scale tests on single elements. Within a caisson filled with a loose sand stratum obtained by means of a pluvial sand deposition system, a prototype sand column (40 cm high and 4 cm of diameter), circumferentially reinforced by means of one geosynthetic, has been created. The sand column has been obtained by means of manual sand tamping. In order to study the effect of the reinforcement stiffness on the foundation structure, different types of geosynthetics have been tested. Even the effect of the loading history applied on the top of the sand model column via a rigid plate is taken into consideration during the tests. Many different unloading/reloading cycles were performed at different loading levels and with different amplitudes. It has been shown that a previous loading of the column allows to reduce severely the settlement suffered by the system. This is due both to the strains accumulated by the soil within the column and to the stresses accumulated within the reinforcement during the "virgin" loading phase. In particular the complete unloading has shown to cause almost a total relaxation of the stresses accumulated within the reinforcement, thus reducing the benefice given by the previous loading phase. A partial unloading (smaller than 70% of the previous maximum vertical load) will produce a remarkable increase in the overall stiffness during the reloading. This seems to imply that, by playing on the loading process, it would be possible to use these elements as foundation substructures. The experimental data have been numerically simulated by means of a non-linear simplified model capable of taking into account both the previously cited effects. The model assumes the shear stresses to be negligible on each side of the reinforcement. The model simulations are satisfactory from a qualitative point of view and allow to highlight many aspects of the mechanical response of the system.

1 INTRODUCTION

In the last thirty years the use of sand columns has become one of the most widely adopted techniques to reduce the settlements of foundations of embankments on very fine and compressible strata. They have the double function of improving the overall mechanical properties of the stratum and to work as vertical drains, thus reducing the consolidation time. A further improvement can be obtained by adopting the so called geo-reinforced sand columns or Geotextile Encased Columns (GEC). This latter takes advantage of the confining effect on the inner granular material given by the presence of the geotextile. In the last ten years many experimental and numerical researches investigated the behaviour of a GEC (Raithel et al., 1999; Raithel and Henne, 2000; Madav et al., 1994; Malarvizhi and Ilamparuthi, 2004), and usually GEC design is generally based on simplified abaci like those proposed by Raithel and Kempfert (2001) or recently by Alexiew et al. (2003). GEC are nowadays worldwide used in order to minimize the settlements and the number of the drilled columns. This paper takes into consideration a single pile founded on a rigid stratum drilled in a compressible soil layer (Figure 1a). The aim of the work is to study the mechanical behaviour of the system, by taking into account the soil-geotextile interaction. The dominant phenomenon which characterizes the mechanical response of this foundation system is the bulging taking place at small depths (Figure 1b).

Both empirical small scale tests described in the following, and numerical simulations obtained by means of a simplified soil-geotextile interaction model (briefly outlined by the authors in § 3) show clearly the importance of the geotextile presence in preventing bulging from occurring.

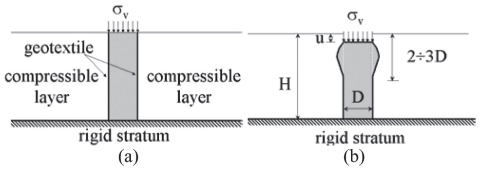

Figure 1. (a) Sand column on a rigid stratum; (b) bulging.

2 EXPERIMENTAL TESTS

Within a rigid square caisson (mm $200 \times 200 \times 400$) a homogeneous loose sand stratum was created by means of the pluvial deposition method. Ticino river sand was used whose properties are in Table 1, and whose grain distribution curve is shown in Figure 2.

Table 1. Geotechnical properties of the Ticino river sand.

Property.	Value
Maximum unit weight	γ_{max} = 16.28 kN/m^3
Minimum unit weight	γ_{min} = 13.80 kN/m^3
Unit weight (loose)	γ_{loose} = 14.58 kN/m^3
Unit weight (dense)	γ_{dense} = 16.48 kN/m^3
Relative density (loose)	Rd_{loose} = 37.2%
Relative density (dense)	$Rd_{dense} \approx$ 100%
Friction angle (loose)	ϕ'_{loose} = 32.35°
Friction angle (dense)	ϕ'_{dense} = 42.12°

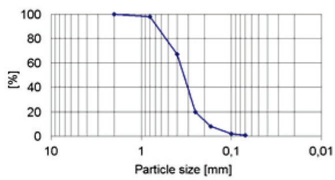

Figure 2. Grain size distribution curve for Ticino river sand.

2.1 *Sample preparation*

A small scale column prototype (diameter D = 40 mm, height H = 400 mm) was realised by means of a rigid pipe encased with the geotextile (Figure 3a).

After the deposition, the sand within the pipe was pumped out (Figure 3b), and the sample column was created by deposing and compacting 2.5 cm of sand at each step for the whole height of the column. The desired compaction degree was obtained by means of an ad hoc soil tamping device. At each step, the rigid pipe (1 in Figure 3c) was pulled out of 2.5 cm thanks to a pipe extractor (2 in Figure 3c).

The soil tamping procedure was carefully chosen in order to get the maximum difference between the relative densities of the sand inside and outside the pile. The adopted procedure leads to a sand density of about 100% within the pile and about 65% outside, in the proximity of the pile.

2.2 *Geotextiles employed*

Four different commercial geotextiles were tested. Their mechanical properties are listed in Table 2.

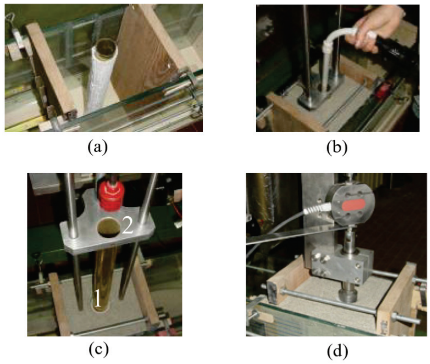

Figure 3. (a) Testing box and rigid pipe; (b) sand aspirator; (c) tamping and pipe extraction; (d) load system.

Table 2. Tensile strength and stiffness of the geotextiles.

		$T_{g,max}$	J
(1)	MACRIT GTV 75-75-D	75 kN/m	625 kN/m
(2)	HATE FILTER C10.341	70 kN/m	466 kN/m
(3)	HATE TAPE 60.006 SP	80 kN/m	400 kN/m
(4)	HATE B 250 K4	18 kN/m	22.5 kN/m

Macrit (1) is composed by a nonwoven geotextile, with a knitted square mesh of polymeric fibres about 5×5 mm in dimension. Hate Filter (2) and Hate Tape (3) are woven geotextiles made of polyethylene, usually adopted as separating and filtering media, while Hate B (4) is a nonwoven geotextile composed by polypropylene fibers. Geotextiles were sewn and glued as to obtain the circular encasing columns (Figure 4).

Figure 4. Sewing of an encasing column in MACRIT.

2.3 *Experimental results*

The chosen loading path consisted in a series of vertical loading-unloading cycles, performed at increasing values of normalised vertical settlement (u/D = 0.1, 0.2 and 0.5), up to a value u/D = 0.75.

The comparison between two preliminary tests on material 1 illustrates satisfactory both the good reproducibility of the test and the effect of the cycle amplitude on the mechanical response of the system. In Figure 5 the results are shown in terms of normalised settlement u/D and vertical stress on the top of the column, computed as the vertical load divided by the initial cross section area of the column. The tests on

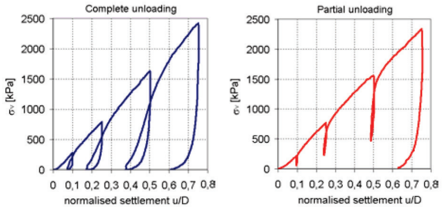

Figure 5. Comparison between complete and partial unloading-reloading cycles for material 1.

the other geotextiles took into account only partial unloading-reloading cycles; in particular we unloaded up to 30% of the maximum vertical load experienced at each cycle. The experimental results concerning the four geo-reinforcements are shown in Figure 6.

As it is quite clear in Figure 6, the curves show a typical locking behaviour for values of u/D smaller than 0.1. By increasing the load, the curves show a change in curvature which can be due either to the seam failure (curves 1 and 3), or to the low stiffness of the geotextile (curve 4). In all cases bulging was observed at the end of the test (Figure 7).

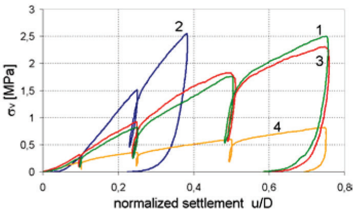

Figure 6. Experimental results.

(a) (b)

Figure 7. (a) Failure of a seam for material 3; (b) bulging in material 4.

3 THEORETICAL MODELLING

In order to reproduce the results of the small scale experimental tests discussed in the previous section, a theoretical interaction model was conceived.

The model interprets the generic slice of height ΔH of the reinforced column as a triaxial sample (Figure 8a). An axi-symmetric reference system is also defined as in Figure 8b. The sample is subjected to a vertical stress σ_v^{in} and to a confining stress given both by the surrounding soil (σ_v^{out}) and to the tension

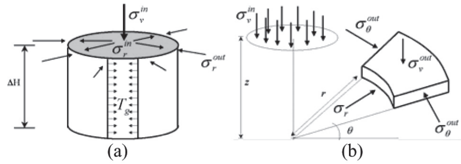

Figure 8. (a) Schematic view of a slice of the column; (b) axi-symmetric reference system.

T_g acting in the geotextile. From the balance equation in radial direction at soil geotextile interface, we can write:

$$\dot{\sigma}_r^{in} = \frac{2\dot{T}_g}{D} + \dot{\sigma}_r^{out} \qquad (1)$$

where dots stand for increments. Equation (1) relies the inner and the outer horizontal stresses to the tensile stress in the geotextile. This latter is supposed to behave as an elastic perfectly plastic material characterized by a tensile stiffness J and a maximum tensile strength $T_{g,\max}$. Outside of the column, the soil is modelled as an elastic perfectly plastic material, characterized by a Young's modulus $E_{s,out}$, a Poisson's ratio ν and by the value of the limit horizontal stress σ_{rL}. This can be evaluated for instance by referring to the pressuremeter test results in granular soil. Inner soil is instead modelled as an elastoplastic material (di Prisco et al., 1993), with a non associated flow rule and an anisotropic strain hardening. As it will be shown in the following, the anisotropy is necessary to capture the loading-unloading response of the system. It is assumed that:

1. the state of stress in the slice is uniform;
2. stresses and strains in radial and tangential directions are equal;
3. shear stress at soil-geotextile interface is absent.

By means of the di Prisco constitutitve model we can rely the stress and strain increments on the slice:

$$\begin{cases} \dot{\varepsilon}_v^{in} = C_{vv}\dot{\sigma}_v^{in} + C_{vr}\dot{\sigma}_r^{in} \\ \dot{\varepsilon}_r^{in} = C_{rv}\dot{\sigma}_v^{in} + C_{rr}\dot{\sigma}_r^{in} \end{cases}, \qquad (2)$$

where C_{ij} are the coefficients of the elastoplastic compliance matrix, while $\dot{\sigma}_v^{in}$ is the known imposed vertical stress increment on the slice. By considering an elastic constitutive law for the geotextile and for the soil outside the column, we can solve the problem incrementally by passing through the definition both of the radial and tangential strains within the column and the definition of the relationship between the radial stress and the radial displacement in the outer part of the domain. By summing up the settlements of all the slices it is possible to obtain the total settlement u of the column, and relate it to the applied vertical load.

3.1 *Numerical simulations*

The results of numerical simulations are shown in Figure 9, where they are superimposed to the experimental data for each geotextile.

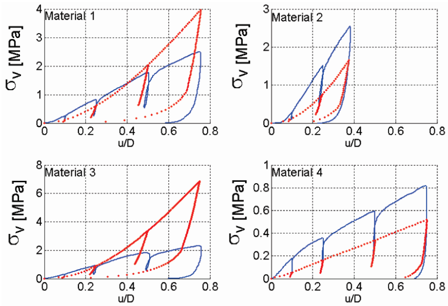

Figure 9. Comparison between experimental data (line) and numerical simulations (dots).

The numerical simulations are quite satisfactory from a qualitative point of view, since the model is capable of taking into account the locking effect given by the geotextile. The unloading phase is correctly reproduced up to a complete unloading, and this is possible only by taking into account the anisotropic hardening of the material inside the column since during the unloading this change in stiffness is due to the fact that the state of stress reaches the yield locus for positive values of the stress deviator. This is clear by considering the effective stress paths of the sand within the column, which, for the sake of brevity, is not reported here. From a quantitative point of view, we must observe that the experimental tests have higher stiffness in the initial part of the loading-settlement curve.

This can be due to the fact that shear stresses at soil-geotextile interface are not taken into account, and the model has not the capability of diffusing the loads (Figure 10). In experimental tests, due to the friction between the soil and the geotextile, the vertical stress in the column tends to decrease with the depth z below the free surface, while in the numerical model it is constant. Moreover, the interaction model does not reproduce the seam failure and the associated bulging, which is responsible of the observed change in curvature in experimental load-settlement curves. Numerical results show instead a continuous locking effect with increasing stiffness, since the yielding in the geotextile or in the outer soil has not yet been reached.

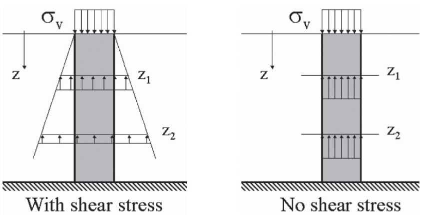

Figure 10. Diffusion of vertical stress within the soil.

4 CONCLUSIONS

The behaviour of georeinforced small scale sand columns under vertical non monotonic loading paths has been investigated. Experimental tests showed how the confining effect given by the geotextile improves both the stiffness and the bearing capacity of the system. During unloading paths, the behaviour of the system is quasi reversible up to about 30% of the previous maximum load; below this level, the system looses the confinement and the overall stiffness decreases. This result suggests that a properly designed preload phase can significantly increase the stiffness of a GEC, so that this type of structure could be employed as foundation system.

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REFERENCES

Alexiew, D., Montez, F., da Silva, A. and Brokemper, D. (2003). "Simplified Estimation and Graphs for Pre-Design of Geosynthetic-Encased Sand or Gravel Columns as Embankment Foundation", pp. 1-6.

di Prisco C., Nova R. and Lanier J. (1993). "A mixed Isotropic-kinamatic hardening constitutive law for sand", Modern Approaches to Plasticity, D. Kolymbas ed., Elsevier App. Sc., pp 83-124.

Madhav, M., Miura, M. and Alagmir, M. (1994). "Improving granular column capacity by geogrids reinforcement", 5th Int. Conf. on geotextiles, geomembranes and related product, Singapore.

Malarvizhi, S.N. and Ilamparuthi, K. (2004). "Load versus Settlement of Claybed stabilized with Stone & Reiforced Stone Columns", 3rd Asian Reg. Conf. on Geosynt., pp. 322-329.

Raithel, M. and Henne, J. (2000). "Design and numerical calculation of a dam foundation with geotexile coated sand columns", Proc. of the 4th Int. Conf. on Ground Improvement Geosystems, pp. 1-8.

Raithel, M., Kempfert, H.-G. and Jaup, A. (1999). "Model tests for analysis of the bearing and deformation behaviour of column foundations", Geot. Eng. for transportation Infrastructure, Barends et. al (ed.), pp. 1521-1526.

Raithel, M. and Kempfert, H.-G. (2001), "Practical aspects of the design of deep geotextile coated sand columns for the foundation of a dike on very soft soils", Landmarks in Earth Reinforcement, Ochiai et. al. (ed.), pp. 545-548.