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Effects of geosynthetics reinforcement on bearing capacity and settlement of shallow foundations

Effets des renforcements avec geosynthétiques sur la portance et le tassement des fondations superficielles

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ABSTRACT
This paper reports the results of model loading tests performed on shallow footings resting on a double-layered deposit of Ticino Sand, a natural silica sand, with geosynthetic inclusions as reinforcement. The sand deposit is reconstituted by raining, using a travelling sand spreader apparatus at purpose developed, in two layers having different relative densities to simulate natural deposits. The soil is pluviated into a steel caisson (1.60x2.50x1.70 m) with reinforced wall; different model foundations, and types and assemblies of geosynthetic reinforcement are installed within. Analyses of the results in terms of bearing capacity, settlement and failure mechanisms are reported.

RÉSUMÉ
Cet article rapporte les résultats des essais de charge réalisés sur fondations superficielles, qui reposent sur un dépot à double couche de sable du Ticino, un sable naturel de silice, avec des inclusions synthétiques comme renforcement. Le dépot est reconstitué par la pluviation à l'air, à l'aide d'un original appareil mobile, en deux couches ayant différentes densités rélatives pour simuler les dépots naturels. Le sol est déposé dans un caisson d'acier (1.60x2.50x1.70 m) aux parois renforcés; dedans on y a installé des différentes fondations-modèle et différents types et montages du renforcement avec geosynthétiques. Des analyses des résultats en termes de portance, tassement, et mécanisme de rupture sont rapportées.

1 INTRODUCTION

Owing to the fact that soil reinforcement techniques by geosynthetics have become useful and rather cost-effective in solving many problems in geotechnical engineering practice, several examples referring to the behaviour of soil with inclusions and to the feasibility of its use in practical application can be found in the literature of the last thirty years or so.

The use of geogrid/geotextile layers could be particularly convenient when the mechanical characteristics of the soil beneath a foundation would suggest the designer in adopting an alternative solution, e.g. a deep foundation. The incorporation of a rough inclusion in the soil improves foundation performance: soil displacement, due to the applied load, mobilizes the resistance at the interface soil/reinforcement and the interaction between soil and inclusion mainly depends on the friction at this interface.

The problem needs to be properly investigated in different ways, analysing both the behaviour at the soil/geosynthetic interface and the global behaviour of the reinforced soil-foundation system. Most of the studies performed on this topic are particularly focused on bearing capacity aspects: the results of model tests show a significant increase in bearing capacity due to the inclusion of various geosynthetic layers as reinforcement.

As far as foundation problems are concerned, the aim of the investigation is particularly devoted to the evaluation of the bearing capacity improvement and stiffness modification in the reinforced soil, considering a particular soil-reinforcement assembly; that is a double-layered soil deposit with reinforcement installed within the denser soil strata or at the interface between the denser and the looser one.

2 PAST STUDIES ON REINFORCED FOUNDATIONS

Starting from the pioneering works by Binquet and Lee (1975) and Akinmusuru and Akinbolade (1981), with reinforcing elements like metal strips or natural fiber strips, many other authors (Guido *et al.*, 1985; Khing *et al.*, 1993; Yetimoglu *et al.*, 1994; Gottardi and Simonini, 1995; Alawaji, 2001) have dealt with this particular aspect of foundation engineering, especially performing model load tests with similar procedure but different geometry, loading configuration, materials and monitoring system.

Generally, all tests have been performed:

- in boxes, with rectangular or square base, made of steel or plexiglas with reinforced walls in order to avoid lateral strain during loading;
- on sand deposits, reconstituted by pluviation and, in some cases, compacted by vibration or tamping;
- reinforcing soil by geogrids or geotextiles.

It has to be remarked that almost all of the abovementioned tests have been performed on homogeneous sand deposits, i.e. without varying relative density with depth, and often using multiple reinforcement layers, unless for studies concerning unpaved roads behaviour.

The significant parameters taken into account from all authors are:

- *u*: distance between bottom of foundation and first reinforcement;
- *h*: vertical spacing of reinforcement layers;
- *b*: reinforcement width;
- *N*: number of reinforcement layers;
- *d*: thickness of reinforced soil ($d = u + [N-1]h$);
- *mechanical properties* of the reinforcing inclusion.

Concerning the analyses of scale-model behaviour, the most useful (non-dimensional) parameter, used to determine the increase in the ultimate bearing capacity in presence of reinforcement, is the Bearing Capacity Ratio (*BCR*):

$$BCR = \frac{q_u^R}{q_u^{UR}} \tag{1}$$

where q^R is the average applied pressure on reinforced soil and q^{UR} the one on unreinforced soil.

Many authors evaluated its trend with respect to the parameters *b*, *u* and *N*.

What it is usually observed by model loading tests is that:

- BCR almost increases with increasing reinforcement width *b* until a limit value, beyond which the effect is negligible (Guido *et al.*, 1985; Khing *et al.* 1993; Yetimoglu *et al.*, 1993);
- BCR almost increases with increasing reinforcement layers number *N* (Akinmusuru and Akinbolade, 1981; Guido *et al.*, 1985; Yetimoglu *et al.*, 1993);
- A reinforcing element reaches its maximum performance for $u/B \approx 0.5$ (Akinmusuru and Akinbolade, 1981; Khing *et al.* 1993).

It is also possible to analyse the effect of reinforcement on the overall stiffness, thus on the settlement by a similar non-dimensional parameter (Khing *et al.*, 1993) defined as Bearing Capacity Ratio for Settlement (*BCRs*):

$$BCR = \frac{q_s^R}{q_s^{UR}} \tag{2}$$

where q_s is the average applied pressure, at a certain settlement rate, smaller than the ultimate one in the unreinforced soil.

Khing *et al.* (1993) took into account settlement rates of 25%, 50% and 75% of the ultimate settlement in unreinforced soil, observing a trend similar to *BCR* with respect to *b/B* and that, for all values of *b/B*, $BCRs \approx 0.7\ BCR$.

3 RECONSTITUTION OF SAND DEPOSITS

It is well known that raining techniques give good results in reproducing samples of granular soils to be used in conventional laboratory tests or in scale models; in order to meet the need of reproducibility with the need of casting large volume samples, a travelling sand spreader apparatus was developed and calibrated, with Ticino Sand (see its parameters in Tab. 1), in the Geotechnical Laboratory of the University of Genoa (Passalacqua, 1991).

By choosing the appropriate settings, calibrated opening width, falling height and travelling velocity (kept constant during deposition), the apparatus can carry out a sand bed, characterized by an uniform relative density, within the range from 25% to 70%. It is worth observing that, by changing the above-mentioned parameters, and thus the deposition intensity during the pluviation process, it is possible to obtain layers with different relative density within the same soil volume.

4 EXPERIMENTAL PROGRAMME AND SETUP

Considering the model load tests performed by different authors in the last about twenty-five years and thinking about possible practical uses of reinforced foundations, it was chosen to investigate new aspects of the topic, that is:

- Presence of two soil layers with different mechanical characteristics with a shallow foundation resting on;
- Presence of a single reinforcement layer within the dense soil or at the interface between the two strata.

This choice was made to simulate the possibility of removing and substituting, or improving, by various techniques like tamping, grouting and the like, the mechanical characteristics of a superficial portion of soil and including a geosynthetic layer as reinforcement in it. Infact, this design solution could be a valid alternative to deep foundations, both in term of cost and time.

The Ticino sand deposit is pluviated, by the spreader apparatus described in the previous paragraph, into a steel caisson with dimensions 1.60(width)x2.50(depth)x1.70(height) m and reinforced walls in order to avoid lateral strain during soil placement and loading. The subsoil with low mechanical characteristics is simulated by a loose sand layer ($D_R \approx 35\%$) 1 m thick, whereas the superficial one by a dense sand layer ($D_R \approx 65\%$) as thick as the footing width, *B*.

The geosynthetic used is a biaxial geogrid, thermally included into two thin sheets of non-woven geotextile, whose main physical properties are reported in the following table.

Table 1. Main parameters of Ticino sand and of the geosynthetic used.

	Ticino sand	
e_{min}	minimum void ratio [-]	0.550
e_{max}	maximum void ratio [-]	0.905
D_{50}	diameter of 50% passing [mm]	0.93
C_u	coefficient of uniformity [-]	1.49
G_S	grain specific weight [-]	2.69
	Geosynthetic	
	maximum strength [kN/m]	30
	elongation at failure [%]	3.5
	strength at 2% of elongation [kN/m]	19.5
	unit weight [g/m^2]	150
	thickness at 2 kPa [mm]	0.65
	opening width [mm]	14x14

Two rigid model strip footings, a U100 and a U200 steel profile, are used with dimensions 10x50 cm (named *mod1*) and 20x100 cm (named *mod2*), so with a fixed shape ratio $L/B = 5$; in this way, plane-strain conditions are assured and the influence of the caisson's walls and bottom is negligible at all. A rough-base condition, in order to increase friction, is achieved gluing a thin sand layer at the bottom of the plate.

The vertical centered load is provided by a hydraulic jack, governed by a pump, and measured by a load cell, embedded between the jack and the footing; the footing displacements are monitored by three LVDTs and the foundation settlement is assumed as the average of the three values at its edge. A global view of the complete test assembly is shown in Fig. 1.

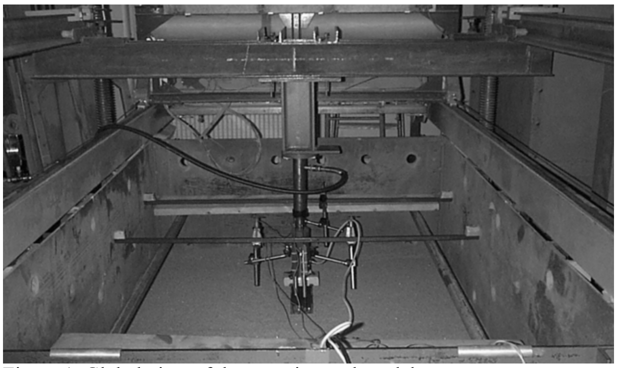

Figure 1. Global view of the experimental model.

All tests are load-controlled, with fixed increment at the different stages and the load is always kept constant until the settlement stabilization. In all the cases, the ultimate condition was well recognized due to the sudden loss of bearing capacity of the system, so the values of load q_u and settlement s_u were easy to be evaluated.

At the end of every reinforced test, and still keeping the applied load, the geogrid position (depth with respect to the ground surface) is measured, in different sections along the longest side of the footing, in order to estimate the difference between the initial and the final configuration and to better clarify the failure mechanism that took place. In this way, it is possible to draw an approximate deformed shape of the geogrid that could help in understanding the mechanism, notwithstanding the difficulty of measuring the load spreading angle through the dense layer.

Thirty-eight load tests have been performed on the two-layered sand deposit, keeping constant the geometrical ratios and using the two abovementioned model-footings. They can be divided into three main series:

- tests on unreinforced deposit (NR1 to NR5);
- tests on reinforced deposit with a single reinforcement layer and different geometrical layouts (RGG1 to RGG33, unless RGG8 and RGG15);
- tests on reinforced deposit with a double reinforcement layer and the same geometrical layout (RGG8 and RGG15).

5 MODEL TESTS RESULTS

A first interpretation of the effects of the reinforcing inclusion can be given by the direct observation of the average pressure-settlement curves, compared to the ones of the tests on unreinforced deposit.

Generally, it can be easily observed, looking at the plotted curves (e.g. Fig. 2), that the insertion of one or two geosynthetic sheet(s) has a significative influence on the behaviour of the system, concerning both the ultimate bearing capacity and the stiffness at different stages, especially at small strain level.

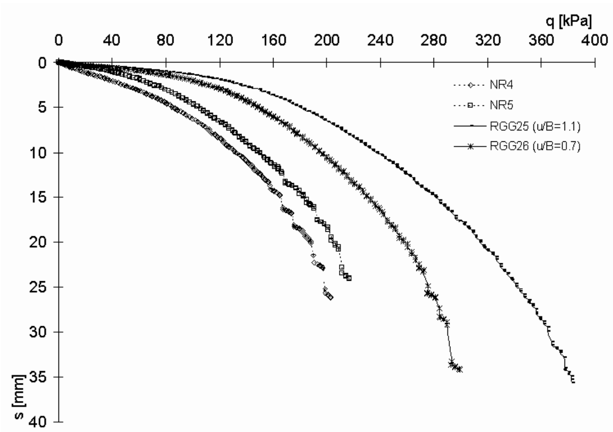

Figure 2. Average pressure-settlement curves (mod2 – b/B=5).

5.1 *Influence of reinforcement width b*

The variation of bearing capacity with respect to the reinforcement width *b* is shown in Fig. 3; it can be noticed that the ultimate load grows with increasing b, independently of the reinforcement depth *u*.

Nevertheless, this trend is more evident in the cases where *u/B* = 0.7 and 1.2, in which the trend lines nearly overlap; in both cases, a maximum value of *b/B*, beyond which there is no further increase in bearing capacity, and almost equals 7, can be determined.

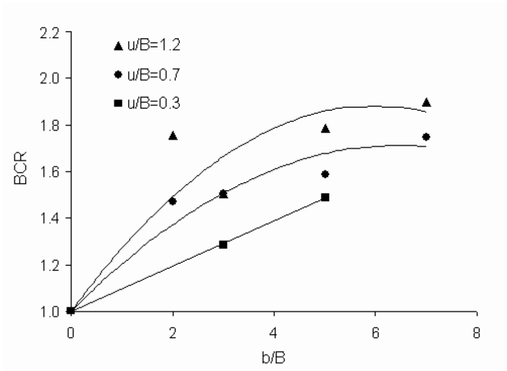

Figure 3. Trend of *BCR* vs. *b/B* for tests with mod1.

The trend is less important in the configurations with *u/B* = 0.3, where the contribution to bearing capacity, provided by the geogrid is lower, as pointed out in the following paragraph. This is in accordance with the results achieved by Akinmusuru and Akinbolade (1981), Guido *et al.* (1985), Khing *et al.* (1993), Yetimoglu *et al.* (1994).

The same remarks can be done also concerning the influence on the stiffness of the reinforced system: fixing a certain value of the settlement rate, the value of *BCRs* grows with increasing reinforcement length *b*.

It has to be pointed out that, from the analysis of the *BCRs* trend, the effect of the inclusion is more significative at low stress/strain level, that is the normal working conditions of real structures.

5.2 *Influence of reinforcement depth u*

It has to be remarked that experimental and analytical studies (Yetimoglu *et al.*, 1994) indicated that the effect of the depth ratio (*u/B*) in single-layer reinforced sand is different from the one in multi-layered reinforced sand, in which the effects of depth ratio and vertical spacing cannot be separated. According to these authors, the optimum depth ratio, at which *BCR* is the highest, is about 0.5 and appears to be independent of the reinforcement width *b*.

The variation of bearing capacity with respect to the reinforcement depth *u* is shown in Fig. 4; it can be observed that a greater efficiency is provided by deeper layers, whereas the effect of geogrids placed at *u/B*=0.3 is less significant.

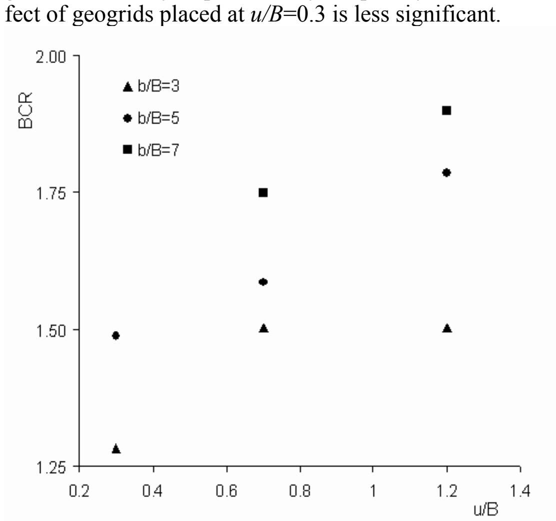

Figure 4. Trend of *BCR* vs. *u/B* for tests with mod1.

5.3 *Other remarks*

The deformed profile of the geosynthetic, measured at the end of every test, gives useful indication about the failure mechanism of the system. Moreover, removing the footing and the sand from their position, in most of the tests it could be observed that the reinforcement layer was visibly deflected in a shape conforming to the size of the footing, suggesting that punching failure through the dense sand had occurred.

In many tests, progressive failure phenomena have been observed, as emphasized by the circles in Fig. 5. Different onsets of failure appear during the test, before reaching a complete instability of the foundation. These phenomena, which increase the overall bearing capacity of the foundation, are related to the reinforcement behaviour that "activates" its shear resistance in correspondence with high soil displacement, thus supporting the loads transmitted by the foundation.

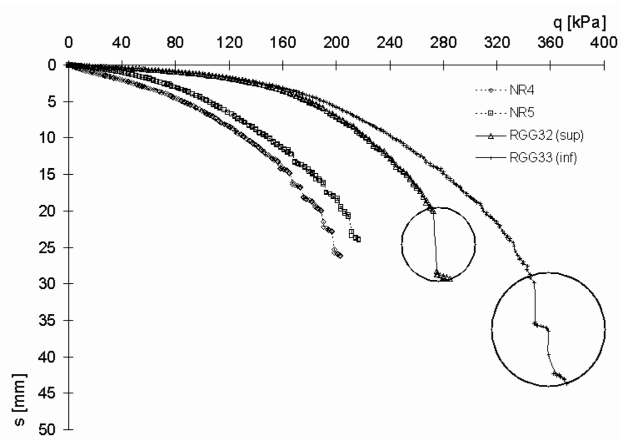

Figure 5. Evidence of progressive failure in the *q-s* curves.

Another mechanism, that is often correlated with the development of progressive failure, is the formation of "slip lines" that appear, at failure, on ground surface in correspondence with geogrid edges (Fig. 6). In this case, it is argued that reinforcement stability could be mainly governed by a "direct sliding" mechanism, more than a "pull-out" one.

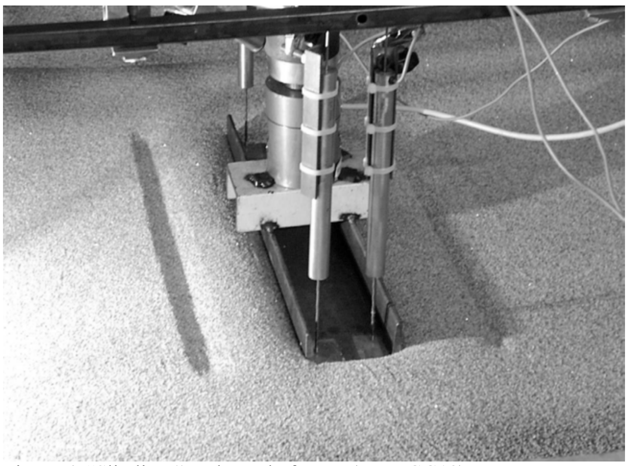

Figure 6. "Slip lines" at the end of a test (test RGG18).

The formation of slip lines, during the performed tests, has been frequently observed for low values of the depth ratio *u/B* ("shallow reinforcement"). When *u/B* increases, slip lines still develop increasing the length of the reinforcement, that is the ratio *b/B*.

As far as bearing capacity of shallow foundations on reinforced double-layered soil deposit is concerned, the only choice of a suitable value of *BCR* does not help the designer in assessing the ultimate load for the foundation; obviously a reliable value of bearing capacity on unreinforced soil is needed.

In the analysis of the performed experimental tests, the application of the solution proposed by Hanna (1981) for a strong sand layer, overlying a weak one, has led to accurate estimates of ultimate average pressures (see Tab. 2).

Table 2. Comparison between experimental and computed ultimate bearing pressures.

Test	Experimental q_u [kPa]	Evaluated q_u [kPa]
NR1	140	134
NR2	140	134
NR3	132	134
NR4	192	251
NR5	208	251

6 FINAL REMARKS

It has to be pointed out that the equipment for the experimental tests do not allow to follow the reinforcement behaviour at different stages, e.g. in terms of deflection and tensile strength: only measurements of the deformed geogrid and ground surface at the end of a test can be carried out. Direct measurements seem to confirm, to a large extent, theoretical indications (see e.g. Giroud and Noiray, 1981).

Hence, further studies by numerical simulation are needed to clarify this topic and to better understand the global mechanisms of the system.

Another point that will be analysed, on the basis of the gained experimental evidences, will be the possible increase of overall resistence, in the denser soil layer under the foundation, due to the "virtual" confining stress exerted by the presence of the reinforcement.

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