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Experimental analyses of geosynthetic reinforced foundations on layered soil deposit

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ABSTRACT: The paper reports the results of experimental analyses performed in order to supply a further contribution on the topic of reinforced foundation performance. As far as experimental tests are concerned, both load tests on model foundations and direct shear tests on reinforced soil are considered. The former to analyse the contribution of geosynthetic reinforcement on bearing capacity and settlements, considering the particular soil-geosynthetic arrangement adopted, the latter to highlight the influence of confining stresses on interface mechanisms. Remarks on the choice of reinforcement geometrical parameters as well as on a possible way of computing ultimate bearing capacity are illustrated and discussed.

1 INTRODUCTION

Since the end of the seventies there has been a great development in reinforcing techniques by using geosynthetics. Nevertheless, a deep knowledge of the phenomena occurring in the reinforced soil is not still gained: empirical rules often used in practice and design procedures reflect this fact.

As far as foundation design is concern, for increasing bearing capacity and reducing settlements, a particular solution could be the placement of reinforcing elements into the soil, in the so called technique of "reinforced foundations". Many studies, carried out in the last thirty years, state that the insertion of rigid or flexible elements beneath a shallow foundation, helps in improving its overall performance. Notwithstanding the rather important contributions to the topic, a lack of a well defined and consolidated design procedure is to be observed.

Considering possible new aspects of the problem and with the aim of supplying further contributions, it was chosen to investigate the behaviour of a multi-layered reinforced deposit, a scheme that is generally adopted for unpaved roads reinforcement. The deposit has been conceived as a upper layer representative of an improved soil portion, (e.g. by compaction), with a geosynthetic layer installed within, on a lower soil layer having poor mechanical characteristics.

The effects of geosynthetic reinforcement have been then investigated by means of plate loading tests, on the so conceived sand deposit, reconstituted in a large test box ($1.60 \times 2.50 \times 1.70$ m) by a travelling sand spreader.

Another aspect that has been investigated is the interface behaviour; direct shear tests have been performed on sand and on sand/geosynthetic interface. The obtained results, useful for the interpretation of the model tests and to perform numerical analyses, show the important role played by interface mechanisms and applied stress levels, that have to be accounted for in practical applications.

2 REINFORCED FOUNDATIONS

Starting from the pioneering works by Binquet and Lee (1975), many other contributions have been given to this particular aspect of foundation engineering.

Most of these have been supplied by the results of experimental tests, performed reinforcing homogeneous sand deposits by single or multiple geogrid or geotextile layers. In some cases, (e.g. for studies concerning unpaved roads) a double soil layer model has been considered.

Significant parameters generally taken into account and adopted in this paper are:

- u: distance between bottom of foundation and first reinforcement;
- b: reinforcement width;
- N: number of reinforcement layers;
- d: thickness of reinforced soil;
- *mechanical properties* of the reinforcement.

A useful non-dimensional parameter, introduced to evaluate the increase in the ultimate bearing capacity in presence of reinforcement, is the Bearing Capacity Ratio (BCR):

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

where q_u^R is the ultimate bearing pressure on reinforced soil and q_u^{UR} the one on unreinforced soil.

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;
- BCR almost increases with increasing reinforcement layers number *N*.

In order to obtain BCR values ranging from 1.5 to 2.5, with usual foundation sizes, typical design parameters are recommended as follows (Wayne et al. 1998): $u/B \leq 0.5$; $b/B \leq 4$; $d/B \leq 2$; $N \leq 5$, being B the footing width.

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

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

where q_s is the average applied pressure, at a certain settlement s, smaller than the ultimate one s_u, in the unreinforced soil (see Fig. 2).

3 EXPERIMENTAL LOAD TESTS

As previously introduced, it was chosen to investigate the effect of the presence of a single reinforcement placed within a dense soil layer or at the interface between this layer and the underlaying one having poor mechanical characteristics.

Tests on model foundations have been performed with the following set-up: the first soil layer, supporting the test plate, is a dense ($D_R \approx 65\%$) sand layer, as thick as the footing width, B ($d \approx 1.1B$). The layer with poor mechanical characteristics, situated below and 1 m thick, is a loose sand layer ($D_R \approx 35\%$). The reinforcement is obtained by placing a single geosynthetic element at relative depths u/B = 0.3-0.7-1.1 (i.e. $u/d \approx$ 0.25-0.6-1.0).

Soil model is constituted by a uniform silica sand (see Table 1), poured by a travelling sand spreader into a large steel caisson (1.60 (width) × 2.50 (length) × 1.70 (height) m). Uniformity in density is obtained by controlling, during deposition, sand falling height, having previously calibrated the opening slot and velocity of the sand spreader.

The geosynthetic used is a biaxial aromatic polyamide geogrid, embedded into a polyester non-woven; it acts as reinforcement and separation; the main physical properties are reported in Table 1.

Table 1. Main parameters of Ticino sand and geosynthetic.

	Ticino sand	
e_{min}	minimum void ratio [–]	0.550
e_{max}	maximum void ratio [–]	0.905
D_{50}	50% diameter [mm]	0.93
Cu	coefficient of uniformity [–]	1.49
G_S	specific gravity of solids [–]	2.69
	EnkaGrid TRC	
	maximum strength [kN/m]	30 × 30
	elongation at break [%]	3.5
	strength at 2% of elongation [kN/m]	19.5
	weight [g/m^2]	150
	thickness at 2 kPa [mm]	0.65
	nominal mesh size [mm]	14 × 14

Two rigid and rough model strip footings, a U100 and a U200 steel profile, with dimensions 10 × 50 cm (*mod-1*) and 20 × 100 cm (*mod-2*), have been used (Figure 1): plane-strain conditions are assured and the influence of the caisson's walls and bottom is negligible at all.

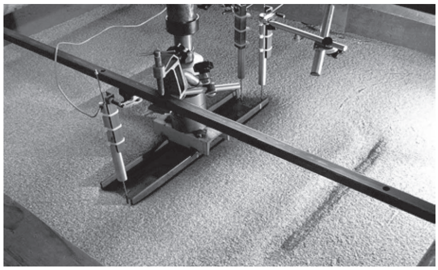

Figure 1. Global view of the experimental model.

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 evaluate the approximate deformed shape of the geogrid that could help in understanding the interaction mechanisms taking place among footing, soil layers and reinforcement.

A total of thirty-three load tests have been performed on the two-layered sand deposit, keeping constant the geometrical ratios and using the two above-mentioned model footings.

Further and more detailed description of the test arrangements and results are reported in Gualco (2005) and Gualco and Berardi (2005). In the following some aspects are briefly summarised.

A first interpretation of the effects of the reinforcing inclusion can be given by the direct observation of the pressure-settlement curves: the insertion of the geosynthetic has always a significant influence on the footing ultimate bearing pressure as well as on the stiffness at different loading stages.

As far as the influence of reinforcement width b is concerned, the obtained results are in agreement with literature indications: BCR increases up to a value of $b/B \approx 5$ ($BC = 1.3 \div 2.2$) then, for longer reinforcements, the ratio remains almost constant.

Concerning the influence of the reinforced system on the foundation stiffness, the value of BCRs grows with increasing reinforcement length b. (see Figure 2). The effect of the inclusion is more significant for small settlements, that is in the serviceability conditions for real structures.

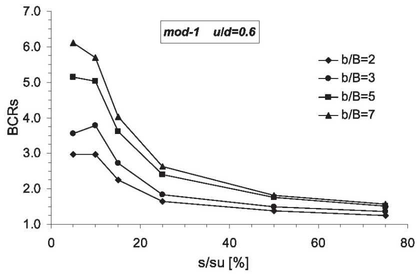

Figure 2. BCRs value for tests on *mod-1* footing.

The variation of bearing capacity with respect to the reinforcement depth u is shown in Fig. 3; it can be observed that, in the performed tests, owing to the particular soil layer arrangement adopted, a greater efficiency is provided by the layer placed at the interface between dense and loose soil strata.

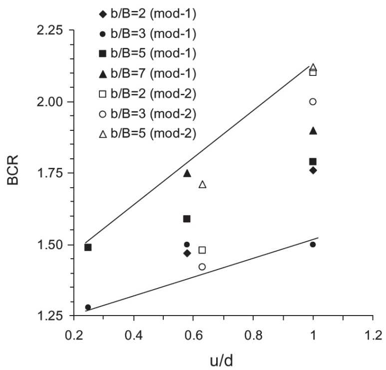

Figure 3. BCR vs. u/d (d: thickness of reinforced soil).

The deformed profile of the geosynthetic, measured at the end of every test, gives useful indication about the failure mechanisms. Moreover, in most of the tests it could be observed, after having removed the load, the footing and the sand, 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 (Fig. 4).

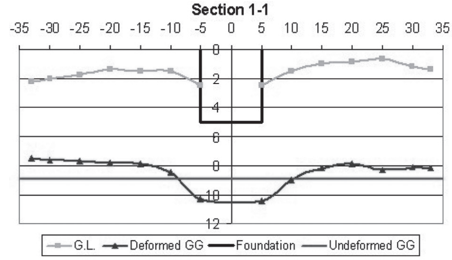

Figure 4. Example of deformed soil-reinforcement.

Values of the load spreading angle seem in good agreement with the value of ~27° (2:1) assumed by Giroud and Noiray (1981).

In many tests, signals of progressive failure phenomena have been observed: different onsets of failure appear during the test, before reaching a complete instability of the foundation. 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. 1). In this case, it is argued that reinforcement stability could be mainly governed by a "direct sliding" mechanism, more than a "pull-out" one.

The formation of slip lines, during the performed tests, has been frequently observed for the lowest values of the depth ratio u/B.

Finally, it is worth observing that the use of BCR values in design analyses relies on the knowledge of unreinforced foundation bearing capacity. In the performed tests, the application of the solution proposed by Hanna (1981), for a strong sand layer overlaying a weak one, has led to quite accurate assessments of ultimate bearing pressure (ratio q_u^{UR}(calc.)/q_u^{UR} (meas.) = 0.95-1.2). On the other hand, the use of the Hanna's relationship, as modified by Wayne et al (1998), could led to an overestimation of the ultimate pressure for reinforced foundation, if the geosynthetic maximum tensile strength is taken into account. Actual interface mechanisms, at failure, have to be considered and the relevant parameters, in terms of soil-geosynthetic shear strength, should be characterized. The following section deals with experimental tests performed for this important task.

4 DIRECT SHEAR TESTS ON INTERFACE

Notwithstanding the uncertainties related to possible critical conditions that could take place during the tests, direct shear tests represent an easy, low-cost and quick tool aimed at analysing soil behaviour by testing small-scale samples. Modified direct shear tests are suitable for simply measuring the coefficient of direct sliding between soil and any type of reinforcing material. In order to evaluate the influence of factors such as the low confining stresses acting on geosynthetics in the reinforced foundations and to have indications on shear strength at the interface, different series of direct shear tests have been performed: tests on Ticino sand (D_R = 40-60-75% and σ_n = 10-25-50-100 kN/m^2) and tests on the interface between Ticino sand and geosynthetic (D_R = 40-60-75% and σ_n=10-25-50-100 kN/m^2).

The results of tests performed on sand supply and confirmed its strength parameters. Owing to the low adopted confining stresses, curved failure envelopes have been observed. The results obtained by interface tests are in good agreement with the ones obtained on sand, as shown in Figure 5. As far as failure loci are concerned, it can be argued that the dependency on relative density and confining pressure is less relevant in the interface tests: they are in fact more markedly linear than the ones obtained in the tests on sand.

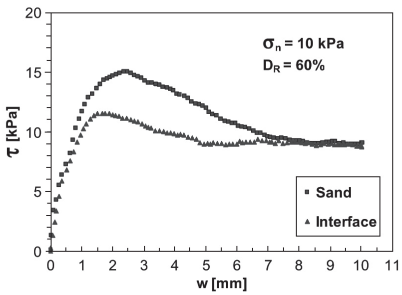

Figure 5. Example of comparison between sand and interface behaviour.

All the analyses performed allow the definition of the interface behaviour, with the evaluation of the parameters to be used. Considering the particular geosynthetic here adopted, it has been assumed that the coefficient of *direct sliding* α_{ds} is equal to the coefficient of *bond* α_b (Jewell 1996).

The ratio τ/σ_n, that provides a direct comparison between the mobilised shear strength in the different tests, at different stress level, has been considered. The ratio τ/σ_n strongly depends on the stress level, as already observed considering failure loci. The difference between the τ/σ_n trends, with respect to D_R, is more evident in the tests on sand, whereas a significant reduction of the dilation influence can be observed in interface tests. A summary of the results, in terms of interface coefficients, are reported in Table 2.

5 CONCLUSIONS

In the paper the results of an experimental investigation on the performance of geosynthetic reinforced foundations are reported.

Table 2. Values of $\alpha_{ds} - \alpha_b$ at different stress levels.

	D_R = 40%	D_R = 60%	D_R = 75%
σ_n =10 kN/m^2	0.98	0.77	0.78
σ_n =25 kN/m^2	0.95	0.77	0.73
σ_n =50 kN/m^2	0.95	0.74	0.75
σ_n =100 kN/m^2	0.95	0.85	0.73

Considering the assessment of bearing capacity of shallow foundations on reinforced double-layered soil deposit (such as the ones modelled in the tests), the analysis of the results gained in the experimental tests allow the following indications:

- failure of the reinforced system can occur by a "deep punching failure" mechanisms; the ultimate bearing capacity can be estimated by the Hanna's (1981) solution as modified by Wayne et al. (1998), considering, as restraining force provided by the geosynthetic, its interface shear strength, governed by the occurring slippage mechanisms and dependent on reinforcement arrangement;
- in this case, particular attention has to be paid in assessing the "efficiency" (in terms of friction) of geosynthetic reinforcement, depending on materials, soil density and confining stresses, as outlined by the results of direct shear testing;
- alternatively, BCR values could be adopted, considering the significant role played by geosynthetic reinforcement geometry, especially in terms of the *b/B* and u*/B* ratios.

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