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A finite element approach to the strength of granular soils reinforced with geosynthetics

Paolo Simonini
University of Padova, Italy

ABSTRACT: Some aspects of an experimental and numerical research concerning the improvement of the mechanical characteristics of sand reinforced with geosynthetics are presented in the paper. In particular, the effects of the introduction of reinforcements on the volumetric behaviour of cohesionless soil are considered and analysed using the finite element method. The finite element model is also applied to the prediction of the bearing capacity of reinforced soil and the results are compared with experimental 1-g model loading tests.

1 INTRODUCTION

The technique of soil improvement based on the use of geosynthetics as reinforcing elements has greatly increased in the recent years. This type of soil reinforcement has been used to improve the mechanical properties of soils behind a retaining wall, in embankment construction or below paved or unpaved roads.

The design of these reinforced earth structures is commonly based on approaches belonging to the category of the limit equilibrium methods (e.g. De Buhan et al., 1989), which provide the evaluation of the factor of safety against failure at the end of the construction stage but not the distribution of stresses, strains and displacements in the reinforced soil mass.

A powerful tool to overcome this drawback is represented by elasto-plastic analysis carried out with numerical methods, such as the finite element method.

The types of finite element approaches used to modelling the behaviour of reinforced soil can be mainly divided in two classes.

The first one considers the separate discretization of geosynthetic sheets and soil layers (e.g. Karpurapu and Bathrust, 1995) whereas the second uses equivalent non-isotropic elasto-plastic materials representing the behaviour of the whole reinforced mass (e.g. Cividini et al., 1994).

In the present research a finite element approach of the first type is presented and discussed.

The aim of the study is focused on the influence of geosynthetic reinforcements on the behaviour of dense sand at failure considering, particularly, the effects of reinforcing elements on the hydrostatic effective stress level and on the dilation rate.

To this purpose, a laboratory investigation consisting in drained triaxial compression tests on sand reinforced with geosynthetic sheets was performed.

The experimental results were compared with data on natural sand and with results obtained by several numerical analyses, performed with a finite element model based on a constitutive model of sand, which can take into account the effects of effective hydrostatic stress level on the volumetric behaviour of soil at failure.

The finite element method was also tentatively applied to the prediction of the bearing capacity of surface footings on reinforced soil and the numerical results were compared with experimental 1-g loading tests, carried out at the University of Padova using a tridimensional small scale physical model.

2. MODEL OF REINFORCED SAND

To model the behaviour of sand an elasto-plastic constitutive law is used.

The failure envelope is a non-linear Mohr-Coulomb surface (Simonini, 1993), extended to three dimensional stress states and expressed by the following equation:

$$F = p' \sin\phi'_{max} + \sqrt{J_2}(\cos\theta - \frac{\sin\theta \sin\phi'_{max}}{\sqrt{3}}) = 0 \quad (1)$$

where $p' = I_1/3$ = mean effective stress and $J_2 = (I_1^2 + 3I_2)/3$. I_1 and I_2 are the first and the second invariants of the stresses tensor. θ is the Lode angle and ϕ'_{max} the maximum friction angle.

The non-linearity of the failure surface is introduced in the strength criterion assuming that the maximum friction angle ϕ'_{max} (in triaxial compression) is function of the constant volume critical angle ϕ'_{crit} and of the maximum dilatancy rate through the following equation (Bolton, 1986):

$$\phi'_{max} - \phi'_{crit} = 10(-\frac{d\varepsilon_v}{d\varepsilon_1})_{max} \quad (2)$$

where $d\varepsilon_v$ and $d\varepsilon_1$ are the axial and the volumetric strain increments in triaxial compression.

The maximum dilatancy rate is a function of of the relative density I_D and of the mean effective stress p' at failure. Aa suggested by Bolton, we assume that the logarithm of mean effective stress effects a linear reduction of the maximum dilatancy rate. Hence, we have:

$$(-\frac{d\varepsilon_v}{d\varepsilon_1})_{max} = \lambda I_D \ln\frac{p'_{crit}}{p'_{min}} \quad \text{for } p' \le p'_{min} \quad (3a)$$

$$(-\frac{d\varepsilon_v}{d\varepsilon_1})_{max} = \lambda I_D \ln\frac{p'_{crit}}{p'} \quad \text{for } p'_{min} < p' < p'_{crit} \quad (3b)$$

$$(-\frac{d\varepsilon_v}{d\varepsilon_1})_{max} = 0 \quad \text{for } p' \ge p'_{crit} \quad (3c)$$

In the above equations λ is a constant positive multiplier and p'_{min} and p'_{crit} are, respectively, the mean effective stresses below and above which the friction angle is assumed to be constant.

The critical mean effective stress p'_{crit}, where no further dilatative effects on soil are observed and some grain crushing occurs, can be determined from triaxial compression tests at high confining stress.

The minimum volumetric stress p'_{min}, below which the dilation rate is assumed to remain at its maximum value, is 150 kPa (Bolton, 1987).

The behaviour of geosynthetic reinforcements is based on an elasto-plastic Tresca constitutive law:

$$G = \sqrt{J_2}\cos\vartheta - \sigma_g^y = 0 \quad (4)$$

where σ_g^y is the tensile strength of geotextile, considered not dependent on the confining stress. It should be observed that this criterion can be seen as a conservative approximation of geotextile behaviour, when inserted in sand.

In the finite element analysis, sand and geotextile behaviours are linear elastic (E_s, ν_s, E_g, ν_g are, respectively, elastic moduli and Poisson's ratios of sand and geotextile) for stress states within the yield surface. At failure they are perfectly plastic and governed, respectively, by the non-linear Mohr-Coulomb criterion with a non-associated flow rule for sand (dilatancy defined by eq. (3)) and by the Tresca criterion with associated flow for geotextile.

3 LABORATORY INVESTIGATION

The granular soil used in the laboratory tests is a medium-fine uniform quartz sand coming from the mouth of the Adige river, Italy, whose characteristics parameters are: mean particle size D_{50}=0.42 mm; specific gravity G_s=2.71; minimum and maximum dry unit weight 13.6 and 16.5 kN/m^3.

The variation of maximum friction angle with mean effective stress at failure is reported in fig. 1 (Simonini, 1987).

The critical mean stress and critical angle were estimated to be p'_{crit} = 6.8 MPa (for I_D=0.85) and ϕ'_{crit}=35.5° (Mazzucato and Ricceri, 1986).

The constant multiplier λ, defining together with p'_{crit} the curvature of the failure envelope, is 0.2.

The elastic modulus E_s of Adige sand was determined using the following relationship:

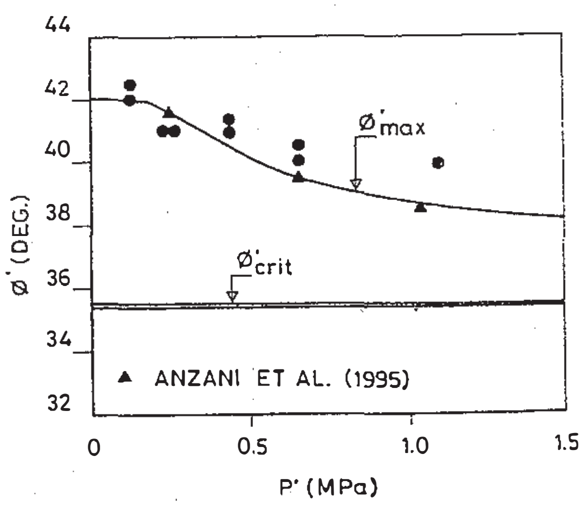

Fig. 1. Variation of maximum friction angle with the mean effective stress at failure

$$E_s = Kp_a(\frac{\sigma'_3}{p_a})^n \quad (5)$$

with K=500, n=0.57. σ'_3 is the confining stress before shearing and p_a is a reference pressure taken equal to 103.33 kPa. Poisson's ratio is ν_s=0.2.

The reinforcement material is a non-woven thermobonded geotextile. The physical properties of the geotextile are: thickness t_{GT}= 0.42 mm and mass per unit area μ=111.5 g/m^2. Tensile behaviour was tested on 100 mm wide - 200 mm long strip (UNI, 1982-1984); the results of 10 elongation tests are shown in fig. 2.

In the finite element analysis the reinforcement is an elastic perfectly plastic material: on the basis of the elongation test results the yield tensile stress and the elastic modulus were selected σ^y_g=6 MPa and E_g=200 MPa. Poisson's ratio is ν_g=0.3.

Drained triaxial compression tests were performed on large specimens (100 mm diameter; 200 mm height) prepared inserting four geotextile sheets spaced at 50 mm. The sand between geotextile sheets was compacted according the technique of "moist tamping" in layers of 25 mm thickness.

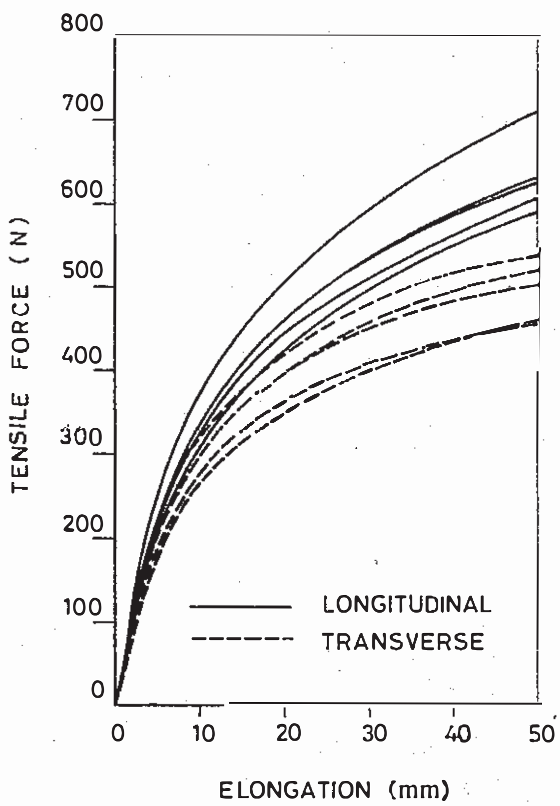

Fig. 2. Results of elongation tests on geotextile.

4 COMPARISON BETWEEN EXPERIMENTAL AND NUMERICAL RESULTS

The results of triaxial compression tests are summarised in table 1, where σ'_{1max} and $(-d\varepsilon_v/d\varepsilon_1)_{max}$ are the maximum axial stress and maximum dilatancy measured at failure.

Table 1. Results of TXCID tests on reinforced sand.

σ'_3 (kPa)	σ'_{1max} (kPa)	$(-d\varepsilon_v/d\varepsilon_1)_{max}$
100	910	0.63
300	1766	0.38
500	2597	0.29

The results of triaxial tests on reinforced sand are also reported on the triaxial plane (fig. 3) together with the data of natural sand (black dots).

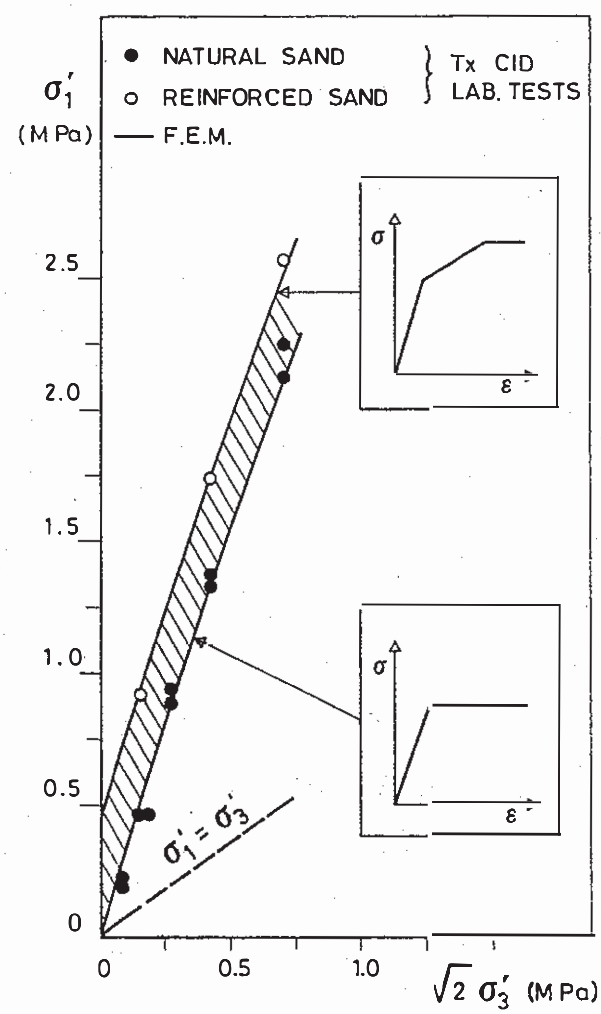

Fig. 3. Experimental and numerical results on triaxial plane

Within the range of confining stress considered in this study, it is shown that the introduction of geotextile sheets leads to a significant increase of the strength at failure of the composite material.

Fig. 4 sketches the values of $(-d\varepsilon_v/d\varepsilon_1)_{max}$ measured in the natural samples. It can be observed that eq. (3) fits the experimental data with reasonable accuracy. The results of tests on reinforced samples are also drawn. In this case, the dilation rate measured form the overall volume change are plotted against the mean stress $p'=(\sigma'_{1max}+2\sigma'_3)/3$, which is assumed, in this context only for sake of comparison, to have uniform distribution inside the sample. Note that the dilation rates $(-d\varepsilon_v/d\varepsilon_1)_{max}$, slightly lower than those of natural sand sheared at the same confining stress, are close to the envelope defined by eq. (3).

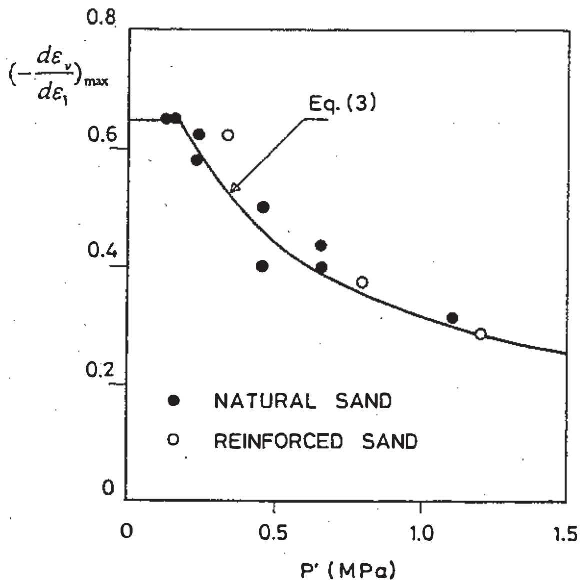

Fig. 4. Maximum dilation rate at failure of natural and reinforced specimens

Finite element simulations were carried out using the constitutive models described above.

The elements used are 8-nodes isoparametric elements with 2x2 gaussian integration rule.

The solution technique is based on the viscoplastic algorithm, whose application to the solution of boundary problems in sand was already described by the author (Simonini, 1994).

From consideration on the symmetry of the problem, only two reinforcements are modelled in the analyses (fig. 5).

It was observed (Fogale, 1993) that the interface friction angle between sand the selected geotextile is close to the internal friction angle of sand.

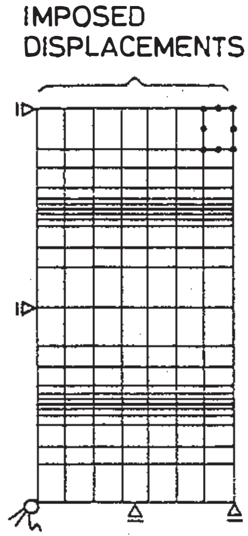

Fig. 5. Finite element mesh

Therefore, no interface elements were used in these numerical analyses. However, future developments will include also this option.

The results of finite element simulations are reported on fig. 3, where they are displayed, for sake of clearness, using continuous fitting lines.

The stress-strain behaviour of natural and reinforced sand, determined from finite element analyses, is schematically represented on the right part of fig. 3. In compression, the first yielding of reinforced soil is followed by a continuous hardening of the material, which is due to the contribution of geotextile sheets. When the geotextile reaches its maximum strength, the composite material flows at constant stress.

It is interesting now to consider the distribution of dilatancy, which depends on the distribution of mean effective stress, in the sample.

Figure 6 sketches the contours of equal dilation rate $(-d\varepsilon_v/d\varepsilon_1)_{max}$, spaced at an interval of 0.1, calculated in the triaxial test simulation with $\sigma_3'=500$ kPa.

The hypothesis of homogeneous deformations would require the presence of the same dilatancy level in the whole specimen.

On the contrary, the effect of reinforcements on sand behaviour is more marked between the geotextile sheets in some parts of the internal zone, where the mean effective stress is higher and, therefore, the dilation rate is lower. On the right border, the confinement effect given by geotextiles is less important and the soil can expand with an higher degree of dilatancy.

5 BEARING CAPACITY ANALYSES

The finite element model was applied to the analysis of the bearing capacity of a circular surface model footing resting on reinforced dense sand.

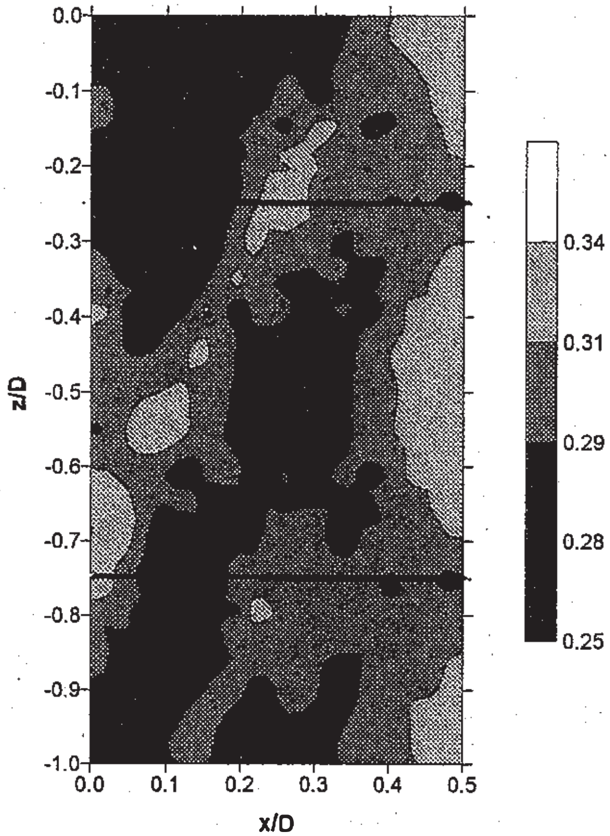

Fig. 5 Distribution of $(-d\varepsilon_v/d\varepsilon_1)_{max}$ in the upper half of the soil specimen tested at σ_3'=500 kPa

The experimental apparatus used to carry out 1-g bearing capacity tests on reinforced soil is a three dimensional physical model which was developed at the University of Padova, with the aim to model the behaviour of surface footings on sand under general loading conditions (Gottardi and Simonini, 1995).

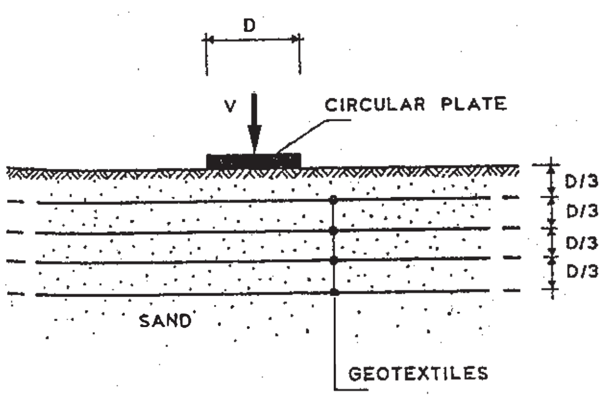

Fig. 6. Schematic view of the bearing capacity problem

Figure 6 provides a schematic cross section of the soil beneath the plate, where 4 geotextiles are placed.

The diameter D of the foundation plate is 150 mm and the vertical spacing between geotextile layers is 50 mm, i.e. D/3. The same type of geotextile used in the triaxial compression tests was adopted for the bearing capacity tests and the sand beds were prepared by pluvial depostion at I_D=85%.

The results of plate loading tests and of finite element analyses are reported in fig. 7.

For sake of comparison, the reference test with unreinforced soil is also drawn on the same figure.

In order to perform the finite element analyses, the knowledge of the foundation soil stiffness was required. In this case, it was assumed that the foundation sand is an homogeneous and isotropic material; its elastic modulus E_S=4.0 MPa (constant with depth) was selected from the back-analysis, carried out using the theory of elasticity (Milovic, 1992), of the load-displacement curve in the loading test with natural sand.

Comparing experimental and numerical load-settlement curves, a satisfactory agreement can be observed. In both cases, the soil-plate failure is of punching type, with continuous load increase necessary to keep the plate moving downwards into the soil.

The finite element analysis show a weaker response of the foundation bed in both natural and reinforced cases during the first stages of loading.

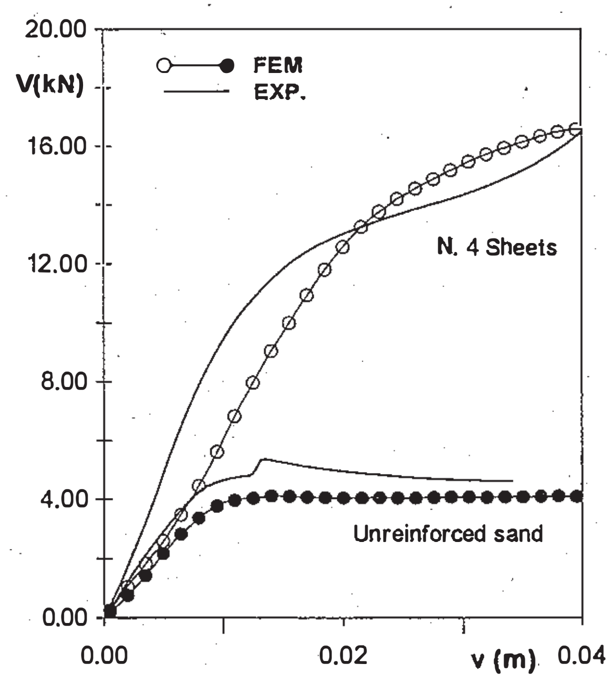

Fig. 7. Load-settlement behaviour in the footing tests.

This effect is more pronounced for reinforced sand, where the influence of geosynthetics on soil stiffness (i.e. increase of elastic modulus of sand) is not properly modelled in the analysis. Future research will consider also this important effect.

6 CONCLUSIONS

A finite element approach to model the behaviour of sand reinforced with geotextiles was presented and discussed in the paper.

To this purpose triaxial compression tests on sand reinforced with geosynthetic sheets were carried out in the laboratory and their results were compared with the results obtained by several finite element analyses, performed with a constitutive model for sand which can take into account the effect of the mean effective stress level on its volumetric behaviour.

Within the range of confining stress considered in this study, it was shown that the introduction of geotextile sheets leads to a significant increase of strength of composite material.

The material models (i.e. for sand and geotextile) used in the finite element analyses have shown to be able to predict with reasonable accuracy the behaviour at failure of reinforced sand in triaxial compression tests. In addition, it was observed that, due to the introduction of reinforcements, the maximum dilation rate is not homogeneous within the sand matrix, but it is lower in the internal zone and higher at the right border of the sample, where the effects of confinement given by geosynthetic sheets are less important.

Finally, the finite element model was also applied to the prediction of the bearing capacity of reinforced soil and the results were compared with experimental 1-g model loading tests. A satisfactory agreement between the overall load-displacement behaviour in both cases was observed especially at higher loads, where the punching failure mechanism begins to develop.

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