Plane strain tests on reinforced sand and their numerical modelling

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ABSTRACT: Some aspects are discussed of the numerical modelling of the load-displacement behaviour observed during plane strain compression tests on samples reinforced with geotextiles. The tests concern samples containing horizontal reinforcements, as well as specimens with various inclinations of the geotextiles with respect to the vertical loading direction. After recalling the basic features of the apparatus, some preliminary experimental results are presented which show a reduction of the overall shear resistance with increasing vertical deformation. Subsequently, the experimental results are compared with those obtained from their numerical (finite element) modelling, based on an "inhomogeneous" approach. Finally, some conclusions are drawn on the influence of the mechanical properties of the geotextiles on the overall behaviour of the tested "composite" material.

1 INTRODUCTION

An increasing interest exists nowadays for the laboratory evaluation of the stress-strain response of natural and reinforced soil samples, in view of the calibration of constitutive laws that can be adopted in the numerical analysis of complex geotechnical problems. In particular, the design of reinforced earth structures can be improved by using ad hoc elasto-plastic material models implemented into non-linear finite element codes. This permits, in fact, to evaluate both the evolution of the strain, stress and displacement fields during construction and the ultimate loading conditions of the completed structure.

The finite element approaches developed to this purpose can be grouped into two main categories, which are referred to as "inhomogeneous" and "non-isotropic" schemes.

When the first one is adopted (e.g. Smith & Segrestin, 1992; Cividini et al., 1997), the reinforced earth structure is discretized considering separately the reinforcements and the soil layers between them. On the contrary, in the "non-isotropic" case the inhomogeneous medium is made equivalent to a continuous, homogeneous, non-linear and non-isotropic material characterised by a suitable constitutive law (e.g. De Buhan & Salencon, 1983; Cividini et al. 1994).

A comparative evaluation of the two schemes (Cividini et al., 1997) was carried out for the case of a vertical earth wall, considering two different kinds of reinforcements, namely: "deformable" oriented polyethylene grids and "rigid" high adherence steel strips.

In the conclusions of that study it was observed that a limited experimental information is available on the stress-strain behaviour of samples containing inclined reinforcements. On this basis it was decided to carry out a laboratory investigation on sand samples containing geotextiles reinforcements with various inclinations with respect to the vertical loading direction.

This investigation, consisting of a series of plane strain compression tests, represents a necessary step towards a deeper understanding of the mechanical behaviour of reinforced sands, in view of the stress analysis of design problems based on the two mentioned finite element approaches.

In the following, the main characteristics of the plane strain device, and of the technique developed for sample preparation, are first summarised. Then, after outlining the ongoing research program, some preliminary test results are presented that show the influence of the reinforcement orientation on the overall load-displacement curves. Finally, the experimental results are compared with those obtained by the numerical simulation of the tests based on an "inhomogeneous" finite element scheme. On the basis of this comparison, some comments are presented on the interaction between sand and reinforcements and on the use of plane strain testing devices for the calibration of constitutive laws for reinforced samples.

2 TECHNICAL CHARACTERISTICS OF THE PLANE STRAIN APPARATUS

The conventional triaxial apparatus, due to its geometrical characteristics, permits to test cylindrical specimens containing only horizontal, or vertical, reinforcements. In addition, only two stress (or strain) components can be independently assigned.

To overcome these limits, various non-conventional testing devices have been proposed in the literature (see e.g. Atkinson et al., 1991; Cornforth, 1964; DeGroot et al., 1993; Reads & Green, 1976). Among them, the plane strain apparatus offers the possibility to test prismatic specimens with reinforcements having various orientations, under three different principal stresses, even though only two of them are independent (Hambly, 1972; Marachi et al., 1981; Oda et al. 1978; Yumlu & Ozbay, 1995). This apparatus is also currently used for investigating the formation of shear bands in homogeneous samples (Viggiani et al., 1994), instead of more complex true triaxial devices (Desrues et al., 1985).

Due to the above features, a plane strain apparatus was adopted for the purposes of the present study. The same apparatus was previously used for testing homogeneous samples of kaolin (Cividini, 1997) and of loose and dense sand (Sterpi, 1997), in view of the experimental calibration of a strain softening constitutive model.

Before discussing the experimental investigation on reinforced sand samples, it seems worthwhile to summarize here the main features of the plane strain device and those of the data acquisition and control system.

The testing device was originally developed by Drescher et al. (1990) and it is commercially available from Geotest Instrument Corp., IL (U.S.A.). It is designed for testing soil specimens having width of 40 mm, length of 80 mm and height up to 140 mm. The specimen, contained within a rubber membrane, is placed on an enlarged base pedestal and it is laterally confined by two vertical rigid walls, 80 mm apart, connected by four tie rods. The axial load is applied through an enlarged upper plate connected to a loading piston.

Two porous stones are mounted on the top and bottom bases and are connected to the drainage lines. These bases and the vertical walls are glass lined and lubricated to minimize the frictional resistance between them and the specimen.

The base platen rests on a linear bearing sled, sliding over a trackway normal to the plane strain direction. This allows for lateral displacements of the sample when shear bands develop. Since one of the vertical walls consists of transparent Plexiglas, it is possible to observe the specimen deformation during the test and, in particular, during the formation of shear bands.

The plane strain apparatus is placed inside a cylindrical cell, filled with non-conducting silicon oil. A standard triaxial test control panel is used to apply the confining and the back pressures. The supply pressure, up to a maximum of 883 kPa, is produced by an air compressor. The assembled cell is placed on a Wykeham Farrance loading frame for carrying out displacement-controlled compression tests.

A series of transducers is located inside the cylindrical cell. They include seven LVDT and seven load cells, denoted respectively by labels "u" and "l" in Figure 1. The LVDTs u_1 and u_5, mounted apart of the loading ram, permit to measure the changes in height of the sample and its tilting. Two pairs of LVDTs (u_3, u_7 and u_2, u_6), mounted horizontally at about 1/3 and 2/3 of the specimen height, record the changes in its width. The volume changes of the sample are determined on the basis of the recorded lateral and vertical strains. This permits to evaluate volume strain also during tests on dry or unsaturated soils. The last LVDT u_4 is used for monitoring the sled movement.

The load cell l_1 and three subminiature load cells (l_5 to l_7) record, respectively, the axial loads acting on the specimen upper and lower bases. The difference between these loads represents the frictional resistance along the lateral rigid walls. The use of three load cells at the specimen base permits to control the load eccentricity during the test.

The compressive stress component in the direction normal to the deformation plane is measured through three subminiature load cells (l_2 to l_4) placed on one of the two rigid vertical walls. Note that this stress component was not recorded in some previous version of this apparatus (e.g. Drescher et al., 1990; Han & Vardoulakis, 1991), thus limiting the information on the stress evolution during the test.

The pore pressure is measured through one piezoresistive transducer (Pw in Figure 1), located outside the confining pressure cell and connected to the lower porous stone. The global volume change for fully saturated specimens can be evaluated by means of the standard triaxial test control panel.

A specifically developed VisualBasic program governs the acquisition of data. The signals of the displacement transducers and load cells, and the corresponding stress and strain components, can be visualized during tests on the computer monitor, thus allowing for a real time control of the test progress. The raw voltages recordings and the processed data are stored on disk for subsequent analysis.

3 SPECIMEN PREPARATION

The samples, consisting of Ticino sand with a relative density of 70%, were prepared according to the "moist tamping" technique (Mulilis et al., 1975), which ensures a sufficiently uniform density distribution throughout them. The sand, with water content of about 5%, was compacted within a specifically designed mould in subsequent layers of constant thickness (Balduzzi & Milani, 1999). The mould can be rotated by varying the angle β from 0° to 90° (see Figure 2), thus allowing to tamp the sand in horizontal layers. At the end of compaction, two steel blades are inserted within the mould along lines 1-C-B and 2-C-D obtaining a prismatic specimen A-B-C-D. The entire mould is then stored in a refrigerator at a temperature of about -80°C for 24 hours.

To ensure comparable values of the thermal contraction coefficients for frozen sand and mould, a transparent polycarbonate (Makrolon) is used instead of other more contractive materials, like Plexiglas.

The frozen sample can be easily handled at room temperature for almost one hour, which is the time required for the assembling operations. They consist in covering the sample with the rubber membrane, securing it between the vertical walls of the plane strain device, installing the instrumentation set and accommodating the plane strain device in the cylindrical cell.

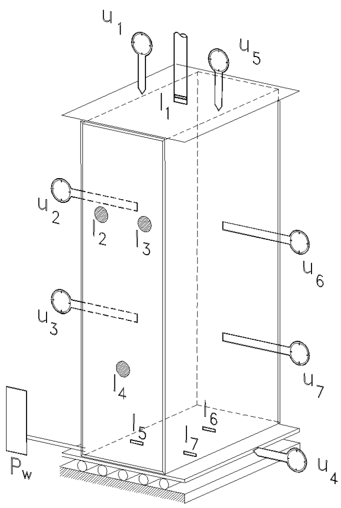

Figure 1. Layout of the instrumentation set used for plane strain tests on prismatic specimens.

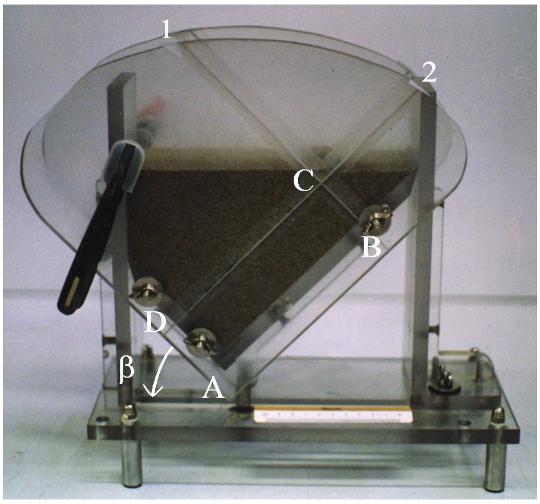

Figure 2. Mould for the preparation of sand specimens with inclined reinforcements (β represents the angle between the reinforcements and the horizontal direction).

After assembling, carbon dioxide and de-aired water are percolated through the sample to remove the air bubbles. Then, the back pressure (of 300.kPa) is applied. The confining pressure is then increased in steps in undrained conditions, obtaining values of the Skempton's pore pressure parameter B greater than .97. After allowing complete dissipation of the excess pore water pressure, the compression tests are carried out at a constant displacement rate of .09 mm/min.

To check the influence of the freezing stage, some standard triaxial and plane strain tests were carried out on samples previously subjected to freezing, considering both the natural sand and samples with horizontal reinforcements. These results were compared with those obtained from unfrozen specimens (Balduzzi & Milani, 1999). Only a minor difference was observed between frozen and unfrozen tests, for all examined cases. This indicates that the freezing process barely affects the experimentally observed stress-strain behaviour.

4 EXPERIMENTAL INVESTIGATION

The experimental program was aimed at verifying the influence on the overall stress-strain behaviour of the orientation, spacing and deformability of the reinforcement (Badiani & Zavanella, 1996; Balduzzi & Milani, 1999; Lüthi, 2000). It consisted of a series of consolidated drained tests, with a confining pressure of 100 kPa. They were carried out on specimens having various values of the orientation of the reinforcements with respect to the horizontal direction (β=0°, 15°, 30°, 45°) and of the geotextile spacing (s=10 mm, 20 mm, 25 mm, 30 mm).

The angle β was kept within the mentioned limits since the specimens with β=45° already show a shear resistance comparable to that of the natural (not reinforced) sand. To quantify the lateral stress influence, two values of the cell pressure (50.kPa and 200.kPa) were used for the samples with β=0°.

In the following, the basic properties of the reinforcement will be first summarized. Subsequently, the results of the plane strain tests will be discussed, together with their numerical analysis through a non-linear finite element model.

4.1 *Reinforcement Characteristics*

The small size of the prismatic samples (40 mm x 80 mm x 140 mm) does not permit the use of the relatively thick geotextiles or geogrids actually used in field applications. Consequently, it was selected a thin polypropylene, spunlaid needled, nonwoven geotextile TS10, produced by Polyfelt Ges.M.B.H. (Austria). This geotextile has a mass per unit area μ=105 g/m^2 and a nominal thickness t_G=1.115 mm (at 2 kPa), determined according to EN 965 and EN 964-1. This choice leads to experimental results that cannot be directly used for the analysis of actual design problem. They permit, however, to define the main characteristics of a suitable numerical model of the composite material, which can be applied also to the analysis of design problems, upon proper calibration of its parameters.

Figure 3 shows the force-elongation diagrams obtained by EN ISO 10319 wide-width tensile tests on 200 mm x 100 mm rectangular samples. The two groups of curves refer to different orientation of the fibers with respect to the direction of load and clearly show the marked anisotropy of the geosynthetic.

Note that the tensile tests lead to underestimated values of the modulus of elasticity and tensile yield limit of the geotextile. In fact, the normal pressure exerted by the soil on the reinforcements, which is not present in the tests, is likely to increase the mentioned mechanical parameters.

Some direct shear tests were also carried out to evaluate the shear resistance of the sand-reinforcement interface (Badiani & Zavanella, 1996). Their results indicate that, in the case of extensible reinforcements, the frictional resistance of the interface is not appreciably different from that of the natural sand. This permits to avoid the use of particular joint elements in the finite element analysis of the laboratory tests.

As shown by a previous numerical investigation (Cividini et al., 1997), the deformability of the geotextile has a significant influence on the overall behaviour of reinforced earth structures. To improve the experimental information on this important aspect, two additional series of tests are presently carried out, using an extensible geotextile and a rather inextensible one.

4.2 *Results of Plane Strain Compression Tests*

For briefness, only the results of six plane strain tests are described in the following. They were performed on samples reinforced with four layers of extensible geotextile, 3.cm apart. Three of them intend to investigate the effects of the lateral pressure on samples with horizontal reinforcements, while the remaining three concern samples with different orientations of the reinforcements.

Figure 4 reports the variation of the axial stress and of the volume strain vs. the axial strain for the first three tests. The confining effect provided by the horizontal reinforcements leads to a monotonous stress-strain behaviour. In addition, the specimens have a dilatant behaviour for low values of the confining pressure.

The influence of the reinforcement orientation (expressed by the angle β) on the axial stress-strain behaviour and on the volume changes is presented in Figure 5. A decrease of the overall stiffness and shear resistance is observed with increasing β, while only a minor difference exists in the volumetric

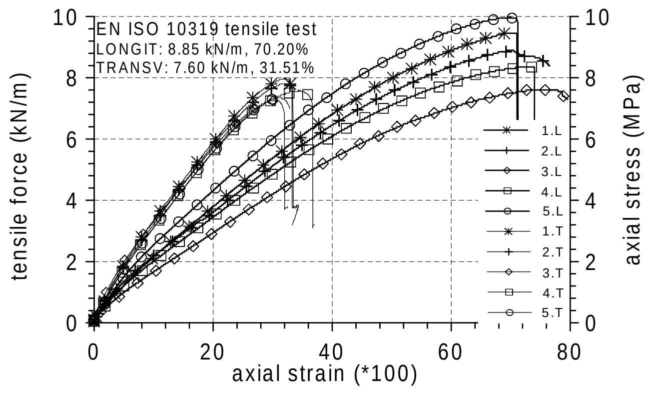

Figure 3. Force-elongation curves obtained by wide strip tensile tests on the geotextile.

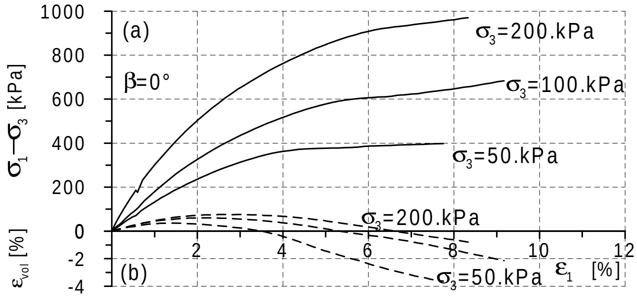

Figure 4. Plane strain compression tests on samples with horizontal reinforcements: (a) axial stress versus axial strain and (b) volume strain versus axial strain.

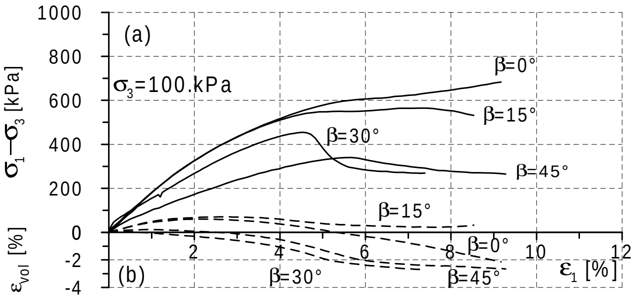

Figure 5. Influence of the reinforcement orientation: (a) axial stress-strain diagrams and (b) volume strains versus axial strain.

behaviour.

The photographs in Figure 6 show that a well-defined shear band develops within a single sand layer when β is equal or greater than 30°. On the contrary, the shear band tends to cross the geotextile layers when β is less than 30°. In addition, a marked strain softening is present for β≥ 30°, while a ductile behaviour is observed when the reinforcements have a lower inclination.

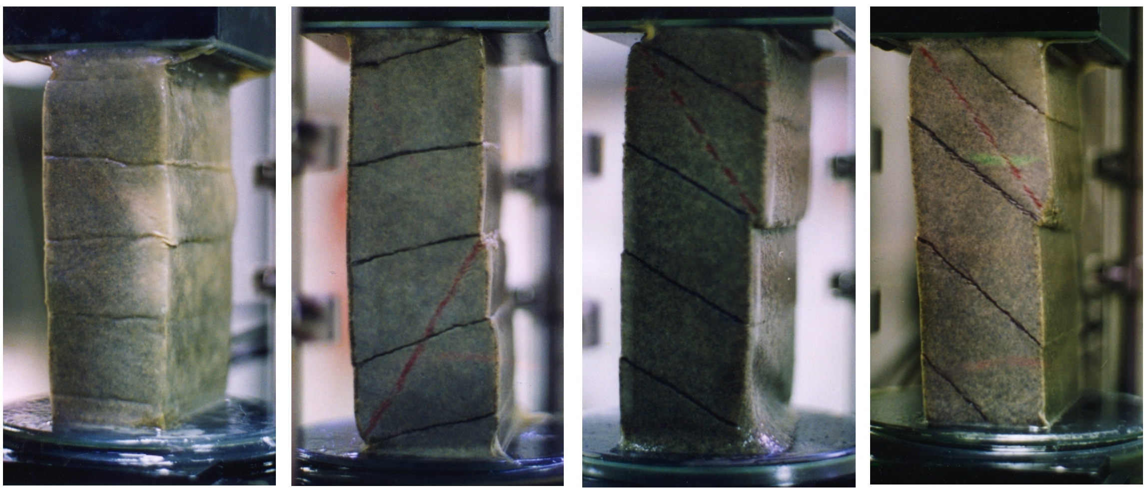

Figure 6. Prismatic specimens after plane strain compression tests.

The results obtained on reinforced samples are compared in Figure 7 with those of the corresponding tests on the natural sand. In particular, the variations are shown of the first stress invariant, I_1, and of the square root J_2 of the second invariant of the deviatoric stress with the angle β. The data refer to both peak and end-of-test conditions. As previously observed, the shear resistance of the reinforced samples almost coincides with that of the unreinforced specimens when β approaches 45°. This implies that reinforced earth structures may give substantially different responses to external load increments leading to appreciable variations of the angle between the reinforcements and the major (compressive) principal stress.

The experimental results indicate also that the overall behaviour of the composite material depends on a variety of mechanical and geometrical parameters, characterising both the basic materials and their assemblage. The numerical analyses discussed in the next Section intend to provide some insight into this behaviour and to single out those properties that have a major influence on overall response of the samples.

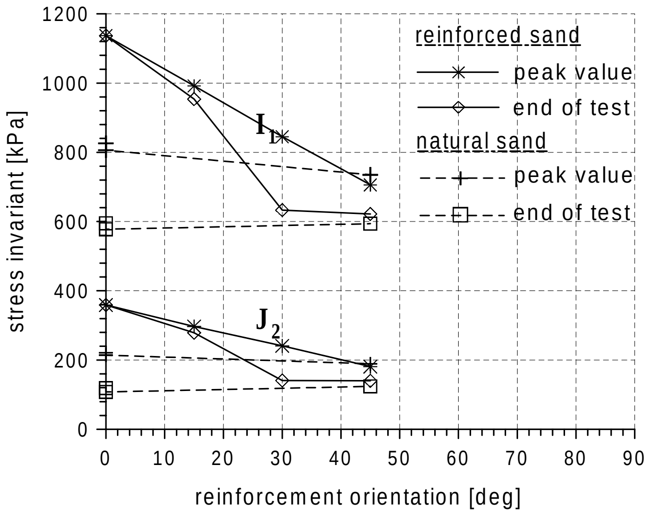

Figure 7. Values of the stress invariants in the reinforced specimens (solid lines) and in the natural sand (dashed lines), at the peak condition and at the end of the test.

5 FINITE ELEMENT MODELLING

The numerical simulation of the plane strain tests was carried out through an "inhomogeneous" finite element approach, i.e. by discretising separately the sand layers and the reinforcements. In particular, the mesh consists of four node, quadrilateral, isoparametric plane-strain elements. Due to the experimental results mentioned in Section 4, no "joint" elements were introduced at the sand-reinforcement interface.

The calculations initiate by imposing the confining stress to the mesh. Then the compression test was simulated by subjecting the top nodes of the mesh to a uniform vertical displacement, which was increased in small steps. The horizontal movements of these nodes were fully constrained. To account for the presence of the sled, only the relative movements of the nodes of the mesh bottom were prevented.

An elastic-ideally plastic behaviour, characterised by Huber-Hencky-von Mises criterion, and a strain-softening model were adopted, respectively, for the reinforcements and for the sand.

The strain-softening constitutive law (Cividini & Gioda, 1992) is based on the assumption that a peak yield condition exists (characterized by cohesion and friction angle) until the irreversible plastic strains attain a given limit. Then, with increasing strains, a decrease of the shear strength parameters occurs, until the ultimate condition is reached.

In spite of the rather simple constitutive model adopted, the numerical results provided an acceptable approximation of the experimental data for the tests on natural sand (Sterpi, 2000), as shown in Figure 8.

As to the reinforced samples, the attention was focused on two aspects (Dei Cas & Zanini, 2000). The first one concerns the modelling of the stress-strain behaviour for axial strain smaller than 4-5%. In fact, the behaviour under relative small strains is crucial for the evaluation of the stress-strain field of actual structures in working conditions, which is mainly governed by the elastic properties.

The second aspect concerns the transition from ductile to strain softening behaviour, with increasing inclination of the reinforcements (cf. Figure 5).

Let discuss first the modelling under small strain conditions. A first analysis was attempted considering the reinforcements as elastically isotropic and deriving their modulus from the linear part of the tensile test curves. The discrepancy between experimental and numerical results turned out to be limited, in the case of horizontal reinforcements, and more pronounced, in the inclined case. However, a proper approximation of the experimental data (cf. Figures 9 and 10) was obtained for both cases considering the elastic behaviour of the geotextile as transversally isotropic. Note that this second set of calculations is still approximated, since it neglects the in-plane anisotropy of the geotextile.

The values of the five elastic constants of the transversally isotropic material were calibrated through a back-analysis of the experimental data. It turned out that the influence of two of them is particularly relevant. The first one is the elastic modulus E_v in the thickness direction. Its back-calculated value is consistent with that derived from the few data available from the determination of the nominal thickness, according to the procedure of code EN 964-1.

A more accurate determination of this modulus can be obtained by subjecting the reinforcements to standard unconfined compression tests. This procedure is suggested as an improvement of the current practice.

The back-analysis of the second relevant parameter (the “out-of-plane” shear modulus G_{hv}) led to a relatively small value. This suggests some further investigation. In particular, it seems necessary to assess if such estimated low value is related solely to the geotextile properties or if, indirectly, it is influenced also by other physical or mechanical effects not considered in the numerical model. For instance, they can be due to the possible anisotropy of the sand layers or to the penetration of the sand particles within the geotextile, that in turn could modify its shear stiffness.

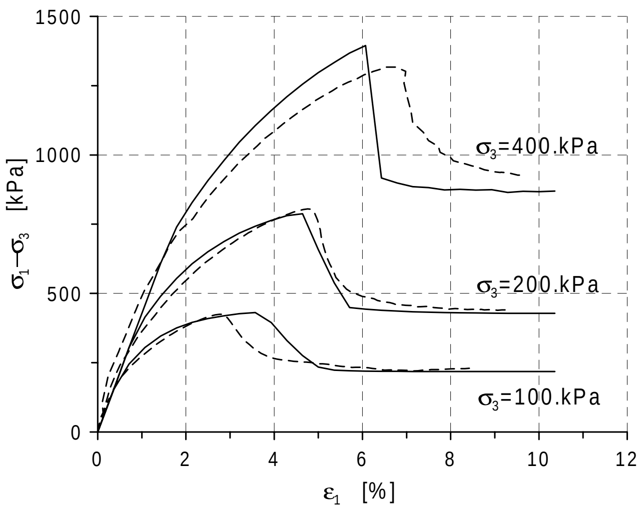

Figure 8. Results of plane strain tests on natural sand (dashed lines) and their numerical interpretation based on the inhomogeneous approach (solid lines).

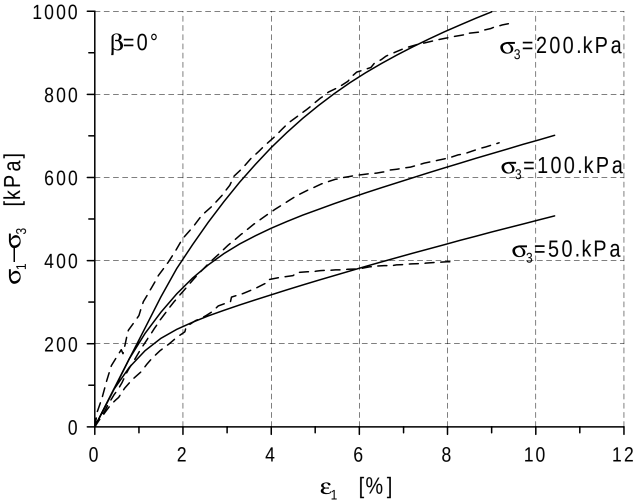

Figure 9. Comparison between the results of plane triaxial tests on horizontally reinforced samples (dashed lines) and of the elasto-plastic finite element analyses (solid lines).

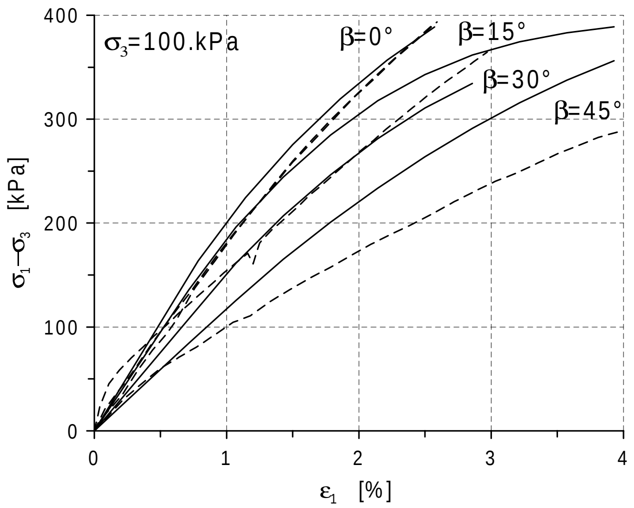

Figure10. Specimens with inclined reinforcements: comparison between the results of plane strain tests (dashed lines) and of the finite element analyses (solid lines).

As to the second aspect of the numerical interpretation, it turned out that the strain softening isotropic model adopted for the sand layers is able to represent the marked influence of the reinforcement orientation on the stress-strain curves and on the failure mode. This is shown by Figures 11 and 12 that report, respectively, the numerically evaluated stress-strain diagrams and the deformed meshes at failure.

A final observation concerns the slope of the softening branch of the sand constitutive law. To reach an acceptable approximation of the experimental results on reinforced samples this slope should be lower than that used for interpreting the tests on the natural sand, and it should be related to the reinforcement inclination. Quite likely, this is due to a non sufficient accuracy in modelling the mechanical behaviour of either sand or reinforcements. Consequently, it seems advisable continuing the numerical investigation introducing, in particular, a non-isotropic plastic behaviour of the components in the strain softening analyses.

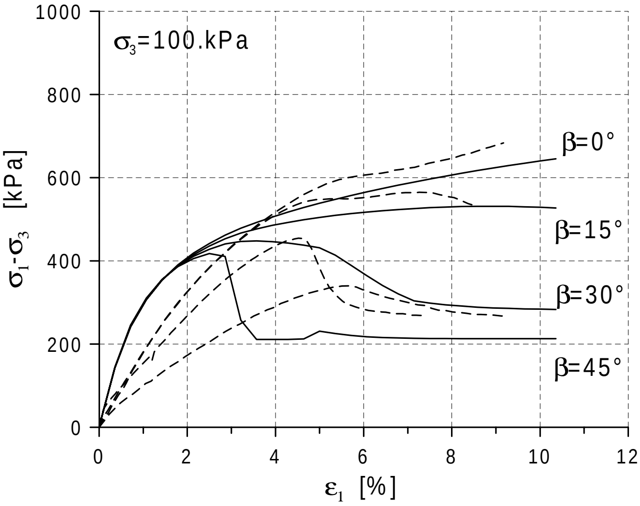

Figure 11. Specimens with inclined reinforcements: comparison between the results of plane strain tests (dashed lines) and of the strain-softening finite element analyses (solid lines).

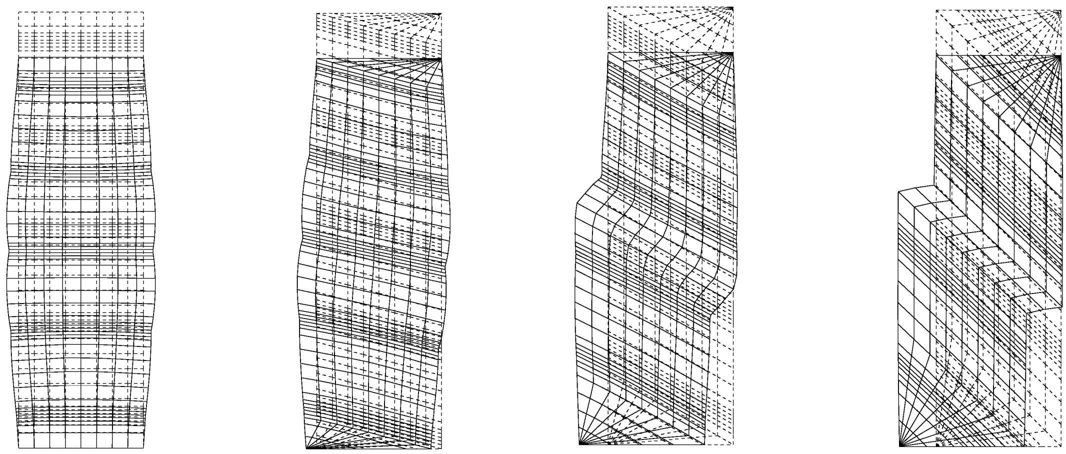

Figure 12. Deformed meshes at failure from inhomogeneous strain softening analyses.

6 CONCLUSIONS

The results of a series of plane strain compression tests have been presented, which were carried out on sand samples reinforced with extensible geotextile layers. They show that the reinforcement orientation has a major role on the overall stiffness and on the shear resistance of the composite material.

The experimental results have been compared with those obtained from a series of finite element analyses, based on an “inhomogeneous” approach. This comparison indicates that the overall stiffness of the samples is influenced by the out-of-plane characteristics of the geotextiles, which are significantly different from in-plane ones. It was also observed that the variation of the overall shear resistance with the reinforcement orientation can be accounted for by adopting a simple strain-softening model for the sand.

To improve the accuracy of the numerical analyses, it seems necessary to extend them towards the use of non-isotropic constitutive laws for the sand and/or for the reinforcements.

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