Sand-Geotextile Interaction by Triaxial Compression Testing

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ABSTRACT: Conventional laboratory triaxial compression equipment is used to conduct tests on sand reinforced with horizontal geotextile discs in order to estimate the apparent angle of friction, δ, between sand and geotextile. The behavior of the composite material (triaxial sample) is also modelled by elastic analysis using an available finite element program. The "equivalent confining stress increase" concept is used to interpret the results obtained from experimental and computer analysis. Observed and computed equivalent confining stress increases due to reinforcement are in very good agreement. The δ values computed from experimental results range between 0.48 and 1.40 of the angle of friction of the sand, decrease with increasing normal interfacial stress and increase with increasing geotextile stiffness. The computer analysis yields a maximum δ value at the perimeter of the reiforcement disc which is in excellent agreement with experimental results.

1 INTRODUCTION

Design procedures for reinforced soil structures require quantification of the interaction behavior at the soil-reinforcement interface which is accomplished by conducting large scale laboratory direct shear and pull-out tests and is expressed in terms of an apparent friction angle, δ(Jones, 1985; Koerner 1990; Ingold, 1994). These experimental procedures require the use of specially designed and constructed large-size direct shear or pull-out boxes and specialized personnel and are rather costly. It is, therefore, of merit to develop a methodology for obtaining δ values from the results of laboratory tests using conventional widely available shear testing equipment. Toward this end, triaxial compression tests were conducted on reinforced sand samples and the experimental investigation was supplemented with results from a linear elastic analysis of similar reinforced sand samples under triaxial loading conditions. Results of this ongoing investigation are summarized herein.

2 EXPERIMENTAL PROCEDURES

Conventional laboratory triaxial compression equipment without modifications was used to conduct tests on geotextile reinforced Ottawa 20-30 sand. All tests were conducted using dry sand with an angle of internal friction of 36° at an average relative density of 87%. Six nonwoven heat bonded geotextiles, all provided by the same manufacturer, were used. Pertinent geotextile properties, according to the manufacturer, are presented in Table 1.

The triaxial compression test samples had a diameter of 76 mm and an overall height of 153 mm and were reinforced with geotextile discs having a diameter of 76 mm. Tests were conducted with a confining pressure, σ_3, ranging from 50 kPa to 400 kPa and at a constant rate of axial displacement equal to 0.6 mm/min. Tests were conducted with samples having the configurations shown in Fig. 1 in order to determine the effect of the reinforcement disc placed at the midheight of the sample. This was dictated by consistent observations indicating that slippage between sand and geotextile occured definitely at least on this reinforcement disc. Reinforced sample configurations similar to that of Fig. 1a have been used previously (i.e. McGown et al., 1978; Broms, 1988) but no attempts were made to separate the effect of one of the reinforcement layers or to estimate values for the apparent friction angle δ.

Table 1 Geotextile Properties (after Williams, 1993)

Geotextiles	Thickness (mm)	Max. tensile load, kN/m	Extension at max. load, %	Load at 5% ext., kN/m
A	0.4	6.1	21	2.7
B	0.7	9.3	25	3.8
C	0.8	11.7	29	4.1
D	1.0	15.1	29	5.1
E	1.2	18.1	33	5.6
F	1.4	21.4	36	6.0

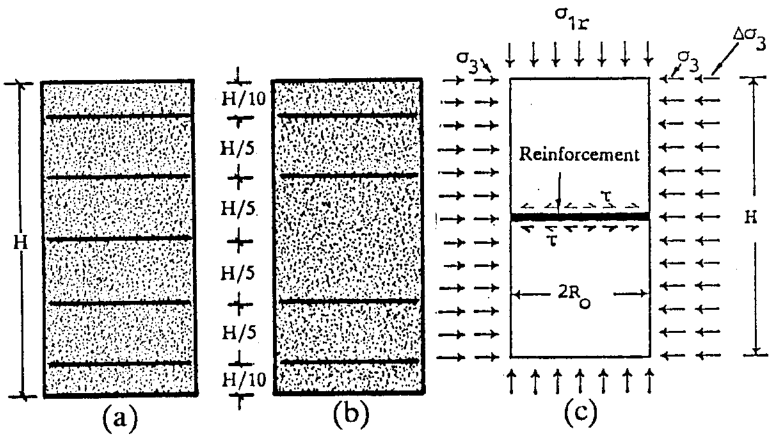

Fig. 1 Reinforced samples: (a) and (b) triaxial testing, (c) failure conditions

3 SAND-GEOTEXTILE FRICTION ANGLE

Typical results obtained by triaxial compression testing are presented in Fig. 2. Both test configurations (five or four reinforcement discs, as shown in Figs 1a and 1b, yielded bilinear envelopes for the composite material which is in good agreement with results obtained by other investigators. The break point of the bilinear evelopes corresponds to normal stress, σ_{vcr}, ranging from 210 kPa to 335 kPa. Available simplified models were extended to facilitate computation of δ.

The "equivalent" confining stress increase, $\Delta\sigma_3$, concept (Ingold, 1982) attributes the observed shear strength increase, due to reinforcement, to the development of an additional confining pressure, $\Delta\sigma_3$, as shown in Fig. 1c for failure conditions. $\Delta\sigma_3$ is considered uniformly distributed over the entire cylindrical surface of the sample and can be expressed, for failure conditions, as (Gray and Al Refeai, 1986):

$$\Delta\sigma_3 = \frac{\sigma_3}{\sigma_1}\Delta\sigma_1 \tag{1}$$

where σ_3 is the same minor principal stress for tests on reinforced and unreinforced soil, σ_1 is the major principal stress at failure of the unreinforced soil, and $\Delta\sigma_1$ is the major principal stress difference at failure between reinforced and unreinforced soil.

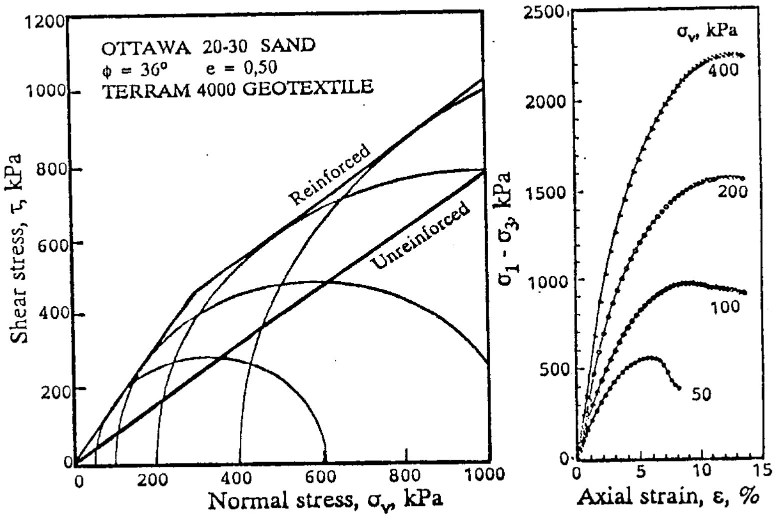

Fig. 2 Typical stress-strain curves and failure envelopes from triaxial compression testing

It has been documented that the shear resistance at the interface is not fully mobilized along the radius of the reinforcement disc (Broms, 1988). By assuming that (a) the mobilized shearing resistance varies linearly along the radius of the reinforcement and (b) the normal stress at failure on the interface at the perimeter of the reinforcement is equal to the major principal stress at failure, σ_{1r}, the distribution of shear stresses along the radius of the reinforcement can be expressed as:

$$\tau = \sigma_{1r}\frac{R}{R_o}\tan\delta \tag{2}$$

where R is the radial distance from the center of the reinforcement disc and R_o is the radius of the disc.

The force, F_δ, acting on the reinforced soil sample shown in Fig. 1c due to the development of shearing stresses on the two soil-reinforcement interfaces should equal the sum, F, of the uniformly distributed confining stress increase, $\Delta\sigma_3$:

$$F_\delta = \frac{4}{3}\pi R_o^2\sigma_{1r}\tan\delta = F = \Delta\sigma_3 2\pi R_o H \tag{3}$$

where H is the overall height of the sample. Accordingly,

$$\tan\delta = \frac{\Delta\sigma_3}{\sigma_{1r}}\frac{3H}{2R_o} \tag{4}$$

The contribution of the geotextile disc at the mid-height of the samples (Fig. 1a) to the shear strength increase was quantified by determining the corresponding confining stress increase, $\Delta\sigma_3$. This was achieved by (a) conducting tests with the same confining pressure, σ_3, for unreinforced sand and sand reinforced as shown in Figs 1a and 1b, (b) determining $\Delta\sigma_{3,5l}$ and $\Delta\sigma_{3,4l}$ for five and four layers of reinforcement, respectively, by applying Eq. (1) and (c) setting $\Delta\sigma_3 = \Delta\sigma_{3,5l} - \Delta\sigma_{3,4l}$. The value of the apparent angle of friction, δ, was then computed by applying Eq. (4) where σ_{1r} was set equal to the axial stress at failure of the sand sample reinforced with five layers of geotextile.

The results obtained are presented in Table 2 normalized with regard to the angle of internal friction, φ, of the sand. The values of δ/φ range generally between 0.48 and 1.40 and indicate a strong dependance of δ on the applied normal

Table 2 Values of δ/φ from Triaxial Tests

Geotextile	Confining Pressure, σ_3, kPa				
	50	75	100	125	150
A	0.98	0.76	0.61	0.50	-
B	0.90	0.69	0.56	0.48	-
C	1.23	0.98	1.03	-	0.64
D	1.40	1.11	0.87	-	0.84
E	1.36	1.14	0.89	-	0.87
F	1.35	1.14	1.13	-	0.83

stress to the sand-geotextile interface. However, the actual interfacial normal stress can not be evaluated reliably at this stage of the investigation. Nevertheless, it can be observed that the value of the apparent friction angle, δ, decreases with increasing normal stress. However, for two of the geotextiles tested, the value of δ/ϕ is consistently below unity indicating, that the interface friction may have never been fully mobilized. Considering that all geotextiles are similar in construction, it can further be observed that the value of δ increases with increasing stiffness of the reinforcement. More specifically, it can be noticed that as the load at 5% deformation of the geotextiles increases from 1.5 kN/m to 7.5 kN/m (Table 1), the δ value changes from 35° to 49° and from 21° to 41° for tests conducted with σ_3 equal to 50 kPa and 100 kPa, respectively.

4 FINITE ELEMENT SIMULATION

Analytical attempts to simulate the behavior of reinforced soil in the entire range of possible deformations, should take into account the non-linear behavior of both soil and reinforcement as well as the interaction at the soil-reinforcement interface. A pertinent finite element model developed by Gray et al. (1989), takes into account geometric and material nonlinearities, allows for the development of slip between sand and geosynthetic material and predicts the stress-strain behavior of geotextile reinforced sand under triaxial loading conditions. However, it should also be recognized that, due to a number of safety factors incorporated into the design of reinforced soil structures, the stress levels expected to develop under working conditions would be many times lower than the ultimate shear resistance of the composite. The behavior of the composite material and its constituents, under low-strain conditions, does not deviate significantly from linearity. Furthermore, it should be expected that no reinforcement slip develops in the low-deformation range. Accordingly, the behavior of the composite material could be modelled by elastic analysis with appropriately selected, strain compatible, elastic moduli. Results from such an elastic analysis, using the SAP 90 program, are presented next.

For lack of space, only results obtained from simulations of Ottawa 20-30 sand reinforced as shown in Fig. 1 with geotextile F(Table 1) and tested under a confining pressure σ_3 = 100 kPa are shown herein. The finite element mesh (901 joints, 208 elements) used for simulating the triaxial sample reiforced with five geotextile discs is shown in Fig. 3a. The elastic modulus of the sand was estimated from the available stress-strain curves and was set equal to 12. 5 MPa. The elastic modulus of the geotextile was estimated by averaging the results of ten wide width tensile tests, provided by Williams (1993), and conducted according to BS6906: Part 1 (ASTM D4595) and was set equal to 86 MPa. Poisson ratio values for sand and reinforcement were assumed to be equal to 0.35 and 0.32, respectively. The deformed shape of the 5-disc reinforced sample, as computed by the finite element program, is shown in Fig. 3b. The axial strain of the composite sample is 7.2 % and compares well with the value of 8.5% observed from the laboratory test (Fig. 2). Furthermore, the overall shape of the deformed sample agrees very well with the post-failure shape observed in the laboratory. For the case of 4-disc reinforced sample, the computed and measured axial strains were 2.6% and 2.4%, respectively. Results of the analysis were also utilized for computing values of the equivalent confining pressure increase, $\Delta\sigma_3$, by distributing the sum of shear forces acting on the sand-reinforcement interfaces over the area of the cylindrical sample. For the cases of 4-disc and 5-disc reinforced samples, the computed values are 36.5 kPa and 187.3 kPa, respectively, and are in complete agreement with the corresponding experimental values (36 kPa, 188 kPa) obtained by use of Eq. (1).

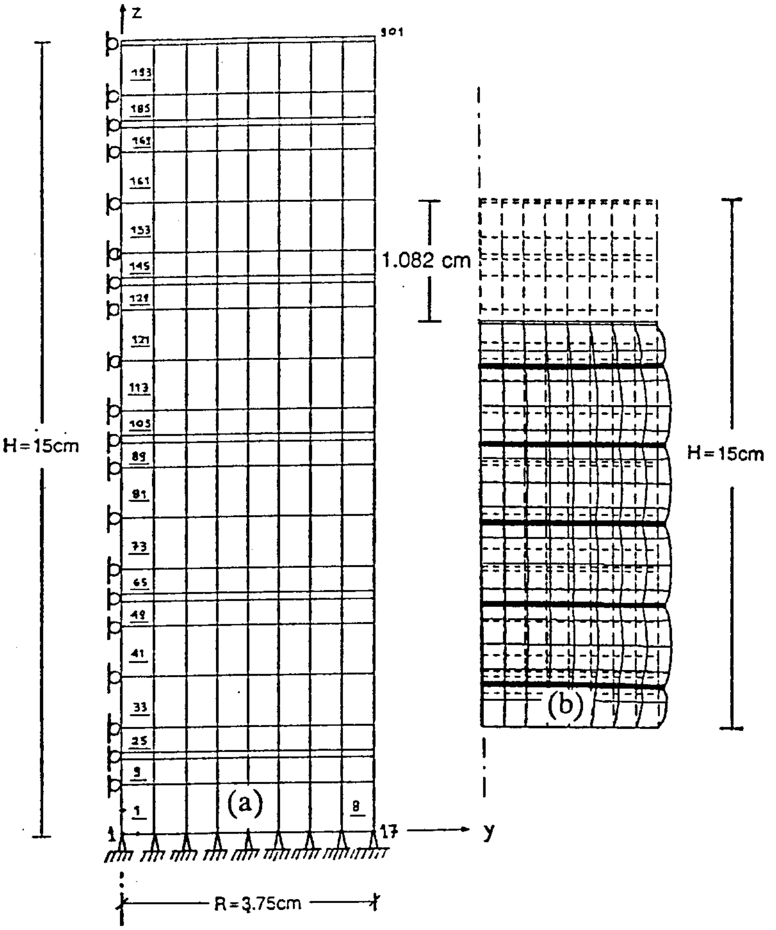

Fig. 3 Computer analysis of sample reinforced with 5 geotextile discs (a) finite element mesh, (b) initial and deformed shape

According to Ingold and Miller (1983), during axial loading of a triaxial sample reinforced with horizontal discs, the shear stress at the soil-reinforcement interface increases, the normal stress becomes progressively lower than the major principal stress and the peak bond stress may or may not develop at a point between the center and the perimeter of the reinforcing layer. The distributions of shear and normal stresses along the radius of reinforcement obtained from the results of the elastic analysis presented herein, are shown in Fig. 4a. It can be observed that the interface shear stress is very low at the central portion of the reinforcement and increases rapidly near the perimeter.

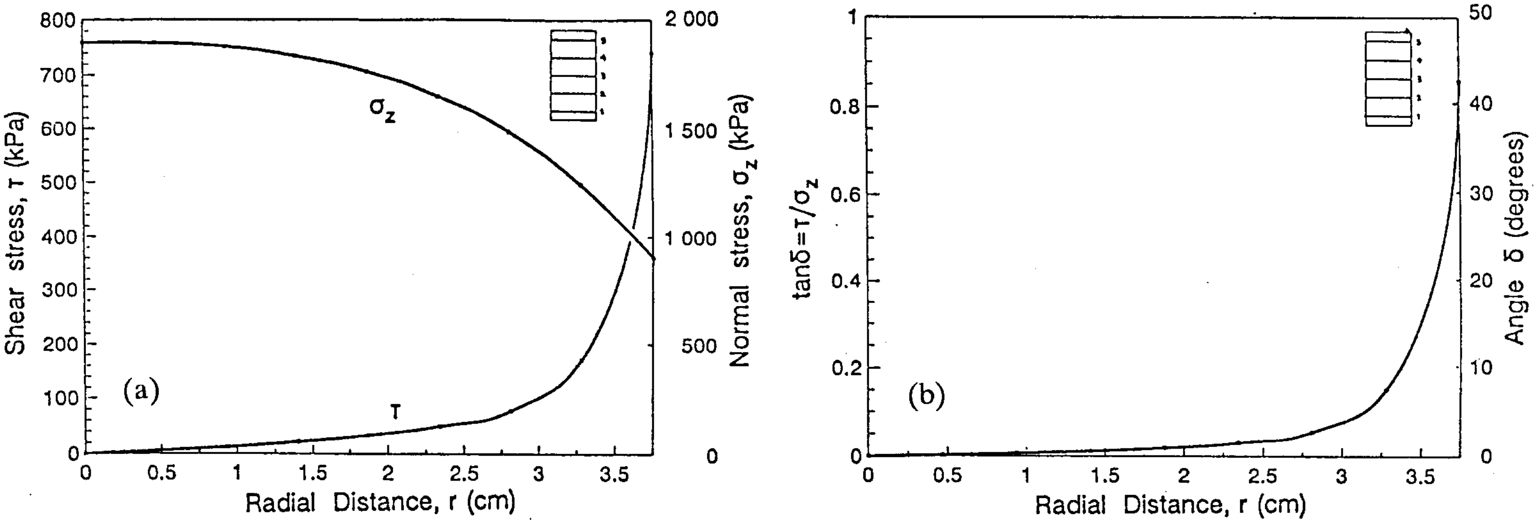

Fig. 4 Distribution of (a) shear and normal stresses and (b) required friction angle, δ, along the radius of the midheight reinforcement

The normal stress is lower than the major principal stress at the perimeter of the sample and higher near the central part of the sample. It should be mentioned, however, that the distributions of these stresses may be changed when an elasto-plastic analysis, instead of an elastic analysis, is conducted. Finally, shown in Fig. 4b is the distribution of the required friction angle, δ, along the radius of the reinforcement for no-slip conditions. The maximum value of δ develops at the edge of the reinforcement (δ = 40.5°) and almost coincides with the value estimated from the experimental results.

5 CONCLUSIONS

Based on the results of this investigation and within the limitations posed by the assumptions made and the materials used, the following conclusions may be advanced:

1. The triaxial compression testing techniques used for estimating values of the apparent interface friction angle, appear to offer a feasible alternative to conventional pull-out and direct shear tests. The results obtained are in quantitative and qualitative agreement with results from other investigations using different testing methods.
2. The value of the apparent interface friction angle decreases with increasing normal interfacial stress and increases with increasing geotextile stiffness.
3. Linear elastic finite element modeling of reinforced soil, using appropriate secant moduli for constituent materials and perfect bond between soil and reinforcement, can simulate satisfactorily the deformed shape of the composite material, the development of equivalent confining pressure increase, $\Delta\sigma_3$, and the required friction bond. However, the distribution of normal and tangential stresses on the soil-reinforcement interface are expected to change when a plastic behavior of soil is postulated.

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