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Prediction of failure stress of reinforced residual soil by simplified approach

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ABSTRACT: A testing program has been carried out to study the stress-strain behaviour of unreinforced and reinforced residual soil. Drained triaxial tests were conducted using computer controlled GDS triaxial apparatus. Triaxial shear test results show that non-woven geotextile reinforced soils exhibit higher failure strains and volume contraction than unreinforced soils. Failure strains and the strength increase with increase in number of layers. A simplified approach for numerical calculations were proposed to predict the shear strength and the coefficient of interface friction of reinforced soils for conventional triaxial compression (CTC) stress paths. Charts were also presented to predict the strength of reinforced soil and to determine the coefficient of the interface friction from triaxial tests. Predictions of failure stress using simplified approach are satisfactory compared to experimental observations.

1 INTRODUCTION

Reinforced soil has gained popularity due to its wide application in the construction of geotechnical structures such as retaining walls, foundations, embankments, pavements, etc. The use of reinforcements increases bond in the soil system due to the interlocking of the soil particles with the reinforcement aperture as well as enhancing the bearing resistance of the transverse members of the reinforcement. The effectiveness of the reinforcements in contributing an increase in the shear resistance is highly dependent on the orientation of the reinforcements with respect to the failure plane. The mechanical and hydraulic properties of the non-woven and composite geotextile have been studied in detail Ling et al. (1990 and 1992). The non-linear stress-strain relationship, which may be highly dependent on the confining stress, was formulated and implemented for finite element analysis by Ling and Tatsuoka (1992). Ling and Tatsuoka (1994) conducted a study on silty clay reinforced with three types of geosynthetics, two geotextiles, and a geogrid under plane strain conditions. The importance of plane strain loading was highlighted, and the influence of the geosynthetic mechanical and hydraulic properties as well as the consolidation stress ratio was investigated in their study. Cuzzafi et al. (1994) were conducted large scale triaxial tests on geogrid reinforced gravel and their test results showed that the apparent cohesion were induced due to the reinforcement in soil. Taha et al. (1999) demonstrated the behaviour of georeinforced residual soil using drained triaxial samples that the reinforced systems increased strength-deformation properties in a significant manner. The results of the failure mechanism indicated strain hardening behaviour and multiple bulging with restraint at the reinforcement layers. Ashmawy et al. (1999) reported that reinforced soils exhibit an improvement in strength and deformation characteristics under monotonic loading conditions, due to the additional "pseudo" confinement caused by the lateral restraint and shear mobilization along the soil-inclusion interface. The research works aimed to determine the stress-strain mechanism between the non-woven geotextile and residual soil by triaxial shear test. However, with respect to residual soil, its interaction mechanism and the failure behaviour in the reinforced composites are not well understood due to limited study. Thus, a thorough investigation of the soil reinforcement interaction was conducted. The simplified prediction procedures to determine the strength of reinforced and unreinforced soils for various stress paths were presented. An attempt was undertaken to determine the coefficient of interface friction from triaxial test results. Prediction charts have been presented for different friction angle of unreinforced soil and number of reinforcement layers. Finally, the equivalent angle of internal friction of reinforced residual soil composites has been derived using the soil friction angle, interface friction and number of reinforcement layers.

2 PROPERTIES OF SOIL AND REINFORCEMENT

Residual soils often found in tropical or semi-tropical area, are formed from intense weathering of rocks under consistently high temperature and rainfall. In this work, the disturbed soil was collected from the area of hospital campus of Universiti Kebangsaan Malaysia (HUKM) located in Cheras, Selangor approximately 8 kilometers south of Kuala Lumpur, Malaysia. The soil is reddish in colour and classified as CH in Unified Classification System (USCS). The soil particle contains about 45 % clay, 19 % silt, 36 % sand and no gravels. The maximum dry density from the standard Proctor test was 14.42 kN/m^3 and the optimum moisture content was about 24.6%.

In this research work, a non-woven geotextiles (Polyfelt TS 60) was used as the reinforcement material. This group of geotextiles consists of mechanically bonded continuous filaments made from UV-stabilized polypropylene. The stress-strain behaviour of five samples of non-woven geotextiles were tested using a universal testing machine. The tensile strength properties of the reinforcement were determined following ASTM D4595-1992 (ASTM 1992) and can be expressed as

$$\sigma_1 = \frac{P_b}{W} \qquad (1)$$

where σ_l is the tensile strength (N/m), P_b is the observed maximum tensile force (*N*) and W is the width of the reinforcement specimen in metre. The maximum tensile strength from the tests was obtained 18.68 kN/m and 19.12 kN/m in the longitudinal and transverse direction, and the corresponding elongations were about 74%, 51% respectively.

3 TESTING PROGRAM

The testing program was performed by a series of consolidated drained triaxial stress path (CTC) tests on unreinforced and reinforced soil. This stress path is followed using the conventional 100mm dia and 200mm high cylindrical triaxial samples. For the CTC stress path test, the incremental stress tensor can be expressed as

$$\Delta\sigma_{ij} = \begin{bmatrix} \Delta\sigma_1 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} \qquad (2)$$

where $\Delta\sigma_{ij}$ is the incremental stress tensor and $\Delta\sigma_1$is the incremental deviatoric stress.

4 TESTING PROCEDURE

In this investigation, six consolidated drained triaxial tests were performed on the unreinforced residual soil. For reinforced soil, twelve tests were carried out for compression stress path with a single layer and two layer non-woven geotextile reinforced specimens. Based on the unit weight and the volume of the triaxial mold, the total weight of the soil was divided into two equal portions for single layer and three equal portions for double layer, and compacted inside the mold in layers of equal height. For a single layer specimen, circular disc of non-woven geotextile was placed at the mid height and for a two layer the distances are 1/3 height from the top or bottom of the specimens. A rate of 0.15 mm/min for compression on a triaxial press was adopted, and each layer was compacted following the approach by Cui and Delage (1996) to ensure Proctor maximum density with a double piston system. The tests reported in this paper for both unreinforced and reinforced soil were carried out under consolidation pressure 100-600 kPa. A strain rate of 0.0015 %/min was used that ensured no pore pressure change as required in a drained test. The computer controlled triaxial (GDS) system was adapted to carry out the CTC stress path tests. A microprocessor collects the data from transducers automatically at prescribed intervals. The data were transmitted by the controlling microprocessor for recording, processing and production of results, which could be displayed on the screen, tabulated or plotted by a plotter.

5 TEST RESULTS

The shear stress vs. axial strain and volumetric strain vs. axial strain curves for unreinforced soil are shown in Figure 1(a) and Figure 1(b). The shear stress-axial strain plot indicate that the axial strain corresponding to maximum shear stress increases with confining pressure. It was obscrved that there were no distinct peak points in the σ-ε curves and the curves levels off at higher strains until failure. The volume change characteristics exhibits contraction behaviour at lower stress levels and expansion at higher stress levels. The cohesion intercept and angle of internal friction of unreinforced soil under compression loading are $c' = 27.42$ kPa, $\phi' = 28.02^{\circ}$ respectively.

For non-woven geotextiles reinforced soil, the stress-strain and volume change characteristics of a single layered soil composites are shown in Figure 2(a) and Figure 2(b) respectively. As expected, the reinforced samples exhibit higher shear strength than unreinforced samples and the maximum shear

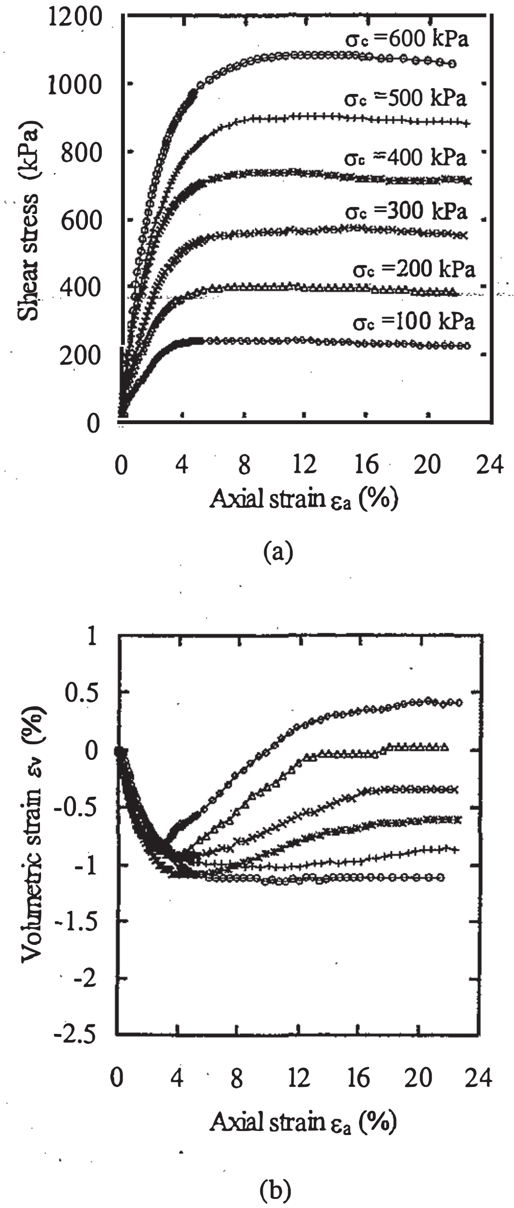

Figure 1. Stress-strain characteristics of unreinforced soil: (a) shear stress vs. axial strain (b) volumetric strain vs. axial strain.

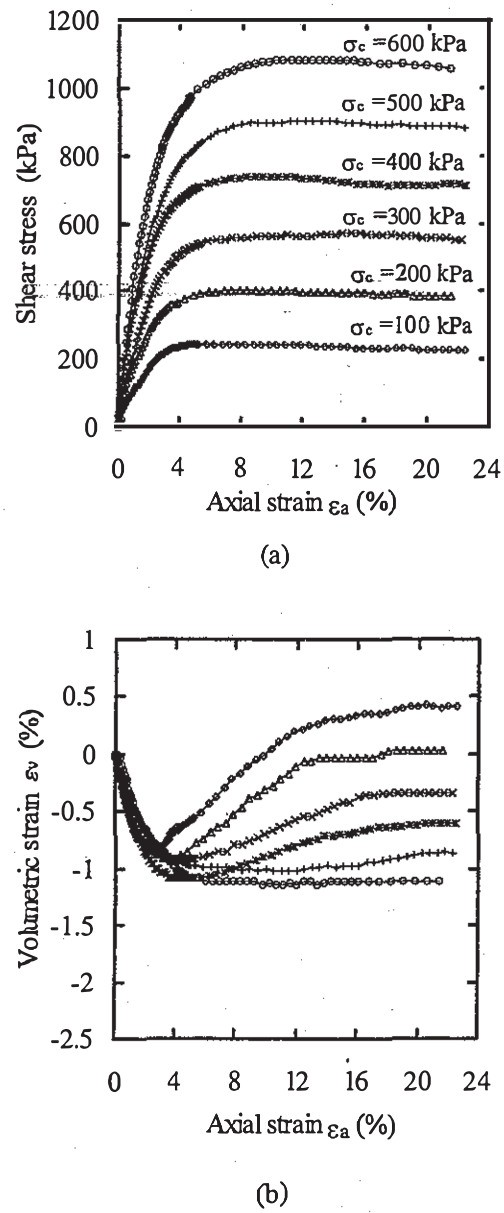

Figure 2. Stress-strain curves of a single layered reinforced soil: (a) shear stress vs. axial strain (b) volumetric strain vs. axial strain.

strength were attained at higher axial strains. This increase of shear strength is caused by an increase of the confining pressure in the soil between the reinforcement layers which depends on the interface friction resistance along the reinforcement.

The shear stress and volume change behaviour of unreinforced and reinforced samples at the initial shearing were similar since the effect of the reinforcement will only begin to function at some finite axial strain. The result shows different volumetric pattern at higher strain when the soil samples started to dilate. From the test results it is also observed that the volumetric strain vs. axial strain behaviour revealed that expansion is more pronounced especially at lower confining pressure. The results of unreinforced and reinforced soil samples also show that dilatancy is dependent on the confining pressure. The shear strength parameters for a single layered and two layered non-woven geotextile reinforced soil are determined from the MIT stress path method. The cohesion intercept and angle of internal friction for a single layered soil composite are $c'=36.99$ kPa, $\phi'=30.76^{o}$ and for a two layered soil are $c'=43.85$ kPa, $\phi'=32.4^{o}$. It is also observed that the reinforced soils exhibit higher failure strain and shows about 20% to 43% higher than that of unreinforced soils.

The failure strains also increases with the increase of reinforcement layers. However, the increase in failure strains over unreinforced soil is not proportional to the number of layer.

6 PREDICTION OF SHEAR STRENGTH BY SIMPLIFIED APPROACH

The prediction of the shear strength of reinforced soils depends on different factors such as the number of reinforcement layer, tensile strength of the reinforcement, confining pressure, and interface friction. Yang (1972) reported for the first time the concept of increase in confining pressure due to insertion of reinforcement. Broms (1977) and Chandrasekaran et al. (1989) proposed various expressions for the shear strength of reinforced soil considering Yang's concept. In this research work, the shear strength expressions for triaxial compression and extension stress paths are derived based on the Yang's concept. The derivation and calculation procedures are outlined in the following section.

According to the Yang's concept, shear stresses developed in the reinforcement layers is assumed to be transferred to the soil through interface friction and this may cause an increase in confining pressure. In this section, an analysis is presented for a single layer of reinforcement placed at the centre of a triaxial sample. During shearing the deviator stress increases and the reinforcement is subjected to tensile stresses which cause the development of interfacial frictional forces between the soil and the reinforcement. The generation of interfacial friction between the soil and the reinforcement are not constant along the reinforcement. Chandrasekaran et al. (1989) reported that the tensile forces in the reinforcement increases from the periphery of the sample to the centre of the reinforcement. The tensile forces in the reinforcement can be expressed as a function of confining pressure and spacing of the reinforcement. The coefficient of interface friction between the soil and the reinforcement can be written as

$$f = tan\delta \tag{3}$$

where δ is the angle of interface friction. The total frictional forces, R_F, at failure along the triaxial cylindrical sample can be expressed as

$$R_F = \frac{\pi}{4}\left(d^2\, \sigma_1\, 2\, f\, K_{av}\right) \tag{4}$$

where σ_1 is the axial stress at failure, d is the reinforcement diameter, K_{av} is the mobilisation factor which is assumed to be equal to $(K_a + K_o)/2$.

The total frictional force, R_F, is assumed to provide additional confining stress effect. Due to the increase in confining stress, the total friction force can expressed as:

$$R_F = \left(\pi\, d\, h\right) \Delta\sigma_3 \tag{5}$$

where h is the height of the sample and d is the diameter of the sample. Combining Equation 4 and Equation 5 the increase in confining pressure can be written as:

$$\Delta\sigma_3 = \frac{K_{av} f\, d\sigma_1}{2h} \tag{6}$$

From the shear strength theory for unreinforced soil the following equation can be written

$$\sigma_1 = \sigma_3 N_\phi + 2c'\sqrt{N_\phi} \tag{7}$$

where $N_\phi = tan^2\,(45 + \phi'/2)$

For reinforced soil, the axial stress, σ_1, can be expressed as

$$\sigma_1 = (\sigma_3 + \Delta\sigma_3)N_\phi + 2c'\sqrt{N_\phi} \tag{8}$$

Substituting the value of $\Delta\sigma_3$ from Equation 6 into Equation 8, σ_1 can written as:

$$\sigma_1 = \frac{\sigma_3 N_\phi + 2c'\sqrt{N_\phi}}{1 - \dfrac{f\, d\, K_{av}}{2hK_a}} \tag{9}$$

For single layer reinforced soil, the axial stress, σ_1 can be predicted from Equation 9 if the values of f, d, h, σ_3 and $\phi'$are known.

A general expression for the axial stress in terms of N layers of reinforcement can be written as

$$\sigma_1 = \frac{\sigma_3 N_\phi + 2c'\sqrt{N_\phi}}{1 - \dfrac{f\, d\, K_{av}}{2hK_a}\displaystyle\sum_1^N\left(1 - \frac{2s}{h}\right)} \tag{10}$$

Equation 10 indicates that the shear strength of reinforced soil is a function of σ_3, ϕ', f, N, (s/h) and (d/h). The coefficient of interface friction has been determined using the failure stress at consolidation pressures σ_c=200 kPa and σ_c=400 kPa for CTC path only. Back predictions of the failure axial stress were then calculated using the average coefficient of friction for the various stress paths. The axial stress of reinforced soil with various types of reinforcement inclusions for CTC stress path is calculated using Equation 10. Comparisons were made with the experimental results. The result shows that the predicted response is a good agreement with the experimental values.

7 PREDICTION OF THE COEFFICIENT OF INTERFACE FRICTION

The coefficient of interface friction of the reinforced soil can determined either from the pull out test or the modified direct shear test. In the field, the reinforced soil is normally subjected to a confining stress. Thus, conducting triaxial tests at the required confining pressure can simulate such conditions. Therefore, an attempt is made to determine the coefficient of interface friction from the triaxial test results. The coefficient of interface friction, f, can be determined by using any stress path test from the values of axial stress at failure, confining stress, angle of internal friction of unreinforced soil, number of reinforcement layers, (s/h) and (d/h), respectively. It is very convenient to determine the coefficient of interface friction, f, by conducting the conventional triaxial compression (CTC) stress path tests. In this stress path the value of f can be determined by rearranging Equation 10.

$$f=\frac{\frac{K_a}{K_{av}}\left[1-\left(\frac{\sigma_3}{\sigma_1}\right)\tan^2(45+\phi'/2)-\frac{2c'\sqrt{\tan^2(45+\phi'/2)}}{\sigma_1}\right]}{\left[\left(\frac{d}{2h}\right)\sum_1^N\left(1-\frac{2s}{h}\right)\right]} \quad (11)$$

For single layer horizontal reinforcement placing at the centre of the specimen, the coefficient of interface friction can be determined as

$$f=\frac{\frac{K_a}{K_{av}}\left[1-\left(\frac{\sigma_3}{\sigma_1}\right)\tan^2(45+\phi'/2)-\frac{2c'\sqrt{\tan^2(45+\phi'/2)}}{\sigma_1}\right]}{\left(\frac{d}{2h}\right)} \quad (12)$$

The above equation can be rewritten when the reinforcement is placed at the mid height of the specimen $(d/h = 0.5)$

$$f=\frac{4K_a}{K_{av}}\left[1-\left(\frac{\sigma_3}{\sigma_1}\right)\tan^2(45+\phi'/2)-\frac{2c'\sqrt{\tan^2(45+\phi'/2}}{\sigma_1}\right] \quad (13)$$

The coefficient of interface friction, f, can be determined from the above equation if the values of σ_1 and σ_3 for reinforced soil and angle of internal friction, ϕ', for the unreinforced soil are known. Prediction charts for finding, f, values are presented in Figure 3 to Figure 6. From the charts the interface friction can be determined if the values of failure ratio (σ'_1/σ'_3), number of reinforcement layers (N) and angle of internal friction (ϕ') for the soil composites are known. In this analysis, equal spacing of reinforcement layers was assumed.

The combined friction angle or equivalent angle of internal friction is one of the important factors which will play a vital role for the improvement of the shear strength of the reinforced soil composites. For the evaluation of the equivalent friction angle, first a relation between the axial stress and confining stress at failure is written down as

$$\sigma_1 = \sigma_3 N_{\phi com} + 2c'\sqrt{N_{\phi com}} \quad (14)$$

where $N_{\phi com} = \tan^2(45+\phi'_{com}/2)$ in which ϕ'_{com} is the combined effective friction angle of reinforced soil. Expanding Equation 14 and rearranging, the combined angle of internal friction of reinforced soil can be determined as:

$$\tan^2(45+\phi'_{com}/2) = \frac{\tan^2(45+\phi'/2)}{\left[1-\frac{f\,d\,K_{av}}{2hK_a}\sum\left(1-\frac{2s}{h}\right)\right]} \quad (15)$$

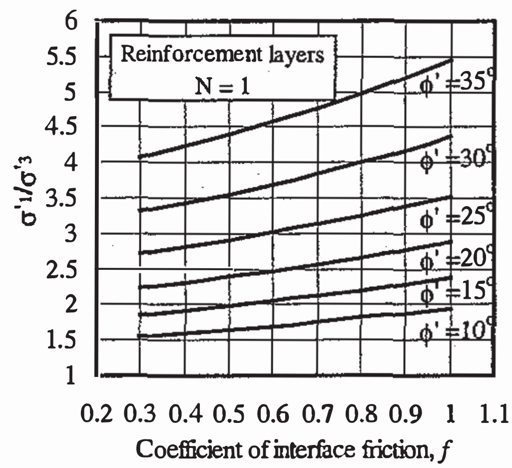

Figure 3. Prediction chart for the coefficient of interface friction for a single layer reinforced soil.

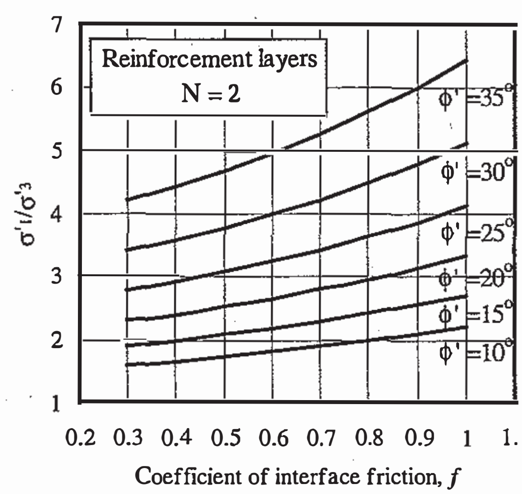

Figure 4. Prediction chart for the coefficient of interface friction for a double layer reinforced soil.

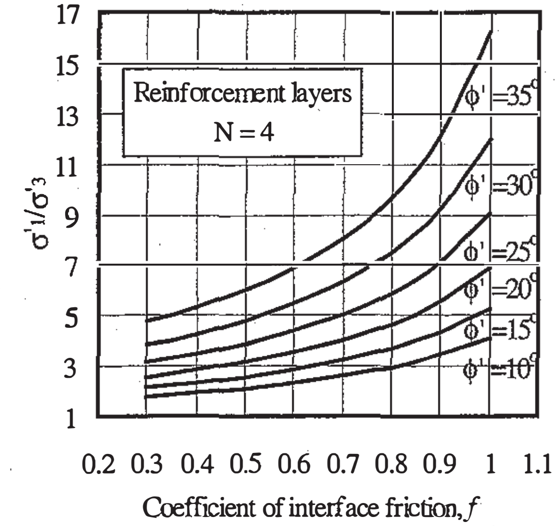

Figure 5. Prediction chart for the coefficient of interface friction for a double layer reinforced soil.

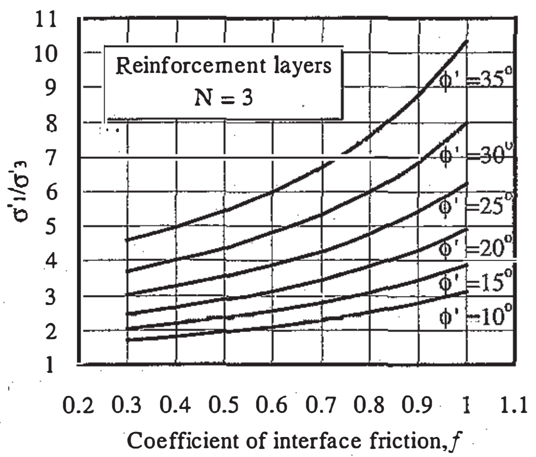

Figure 6. Prediction chart for the coefficient of interface friction for a double layer reinforced soil.

8 CONCLUSIONS

1. Simplified approaches have been proposed for CTC stress paths to simulate the failure stress of the reinforced soil composites.
2. Predictions of the shear strength of reinforced soils can be determined if the angle of internal friction of soil, numbers of reinforcement layer and interface friction coefficient are known.
3. Prediction charts for obtaining the strength of reinforced soil and the coefficient of interface friction are given.
4. A procedure for estimating the combined or equivalent friction angle from triaxial tests data is also presented.

REFERENCES

Ashmawy, A.K., Bourdeau, P.L., Drnevich, V.P. & Dysli, M. 1999. Cyclic response of geotextile- reinforced soil. *Soils and Foundations*, 39(1): 43-52.

ASTM 1992. Testing of geotextiles and related products. *Annual book of ASTM standards*, 04.08. D4595.

Broms, B.B. 1977. Triaxial tests with fabric-reinforced soil. *Proc. Int. Conf. on the use of Fabrics in Geotechnics*, Paris, France, 3: 129-133.

Chandrasekaran, B., Broms, B.B. & Wong, K.S. 1989. Strength of fabric reinforced sand under axisymmetric loading. *Geotextiles and Geomembranes*, 8: 293-310.

Cuzzafi, D., Picarelli, L., Ricciuti, A. & Rimoldi, P. 1994. Behaviour of geogrid reinforced gravel in large scale triaxial tests. *Proceedings of the 13th International Conference on Soil Mechanics and Foundation Engineering, ICSMFE*, New Delhi, India, 1: 271-274.

Cui, Y.J. & Delage, P. 1996. Yielding and plastic behaviour of an unsaturated compacted silt. *Geotechnique*, 46(2): 291-311.

Ling, H.I. & Tatsuoka, F. 1992. Nonlinear analysis of reinforced soil structures by modified CANDE (M-CANDE). *Geosynthetic-Reinforced Soil Retaining Walls*, Jonathan Wu, (ed.), 279-296. Rotterdam: Balkema.

Ling, H.I. & Tatsuoka, F. 1994. Performance of anisotropic geosynthetic-reinforced cohesive soil mass. *Journal of Geotechnical Engineering, ASCE*, 120(7): 1167-1184.

Ling, H.I., Tatsuoka, F. & Wu, J.T.H. 1990. Measuring inplane hydraulic conductivity of geotextiles. *Geosynthetic Testing for Waste Containment Applications, ASTM STP 1081*, Koerner. (ed.), 257-272. Philadelphia.

Ling, H.I., Wu, J.T.H. & Tatsuoka, F. 1992. Short-term strength and deformation characteristics of geotextiles under typical operational conditions. *Geotextiles and Geomembranes*, 11(2): 185-219.

Taha, M.R, Mofiz, S.A. & Hossain, M.K. 1999. Behaviour of georeinforced residual soil in triaxial test. *Proc. World Engineering Congress 99-Towards Engineering Vision: Global Challenges and Issues*, 19th –22nd July, 1999, Kuala Lumpur, 175-180.

Yang, H. 1972. Strength deformation characteristics of reinforced sand. PhD Dissertation, University of California, Los Angeles, California.