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MECHANICAL STRESS-STRAIN CHARACTERISTICS AND MODEL BEHAVIOUR OF GEOSYNTHETIC REINFORCED SOIL COMPOSITES

Syed Abdul Mofiz[1], (Member, ASCE),
Mohd. Raihan Taha[2], (Member, IEM) and Dipopk Chandra Sharker[3], (Member, IEB)

ABSTRACT

Testing of unreinforced and reinforced residual soil were conducted using a computer controlled shear box apparatus with stress levels ranging between 50 to 400 kPa to study its shear strength and deformation characteristics. The effects of reinforcement, orientation and volume changes on the shear strength of soil composites were analysed. From the test results, it was observed that the shear strength and volume change behaviour is related to the degree of reinforcement orientation. It was also observed that the dilation or contraction properties of the soil composites are highly dependent on the applied stress levels. A non-linear elasto-plastic model with plastic hardening was extended to incorporate cohesive soils. The model was then used to predict the behaviour of unreinforced and reinforced soil. The results indicated good agreement with experimental observations particularly at higher normal pressures.

Keywords: Shear strength, Reinforced residual soil, Orientation, Dilation, Model parameters.

INTRODUCTION

Geosynthetic reinforced soil has gained considerable popularity due to its wide application in the construction of geotechnical structures such as retaining walls, foundations, embankments, pavements, etc. Since Vidal first employed it in 1966, significant advances have been made in the design and construction of the system. The use of geosynthetics 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.

It is well known that geosynthetic reinforced soil normally utilises granular soil as its backfill material. Thus, most studies and design methods and charts are well established on the use of such materials. In tropical countries, the locally available residual soil (cohesive material) is too plentiful to be ignored. Furthermore, in terms of cost, the use of locally available materials will

[1]Associate Professor, Department of Civil Engineering, Rajshahi University of Engineering & Technology, Rajshahi 6204, Bangladesh, samofiz@hotmail.com.
[2] Professor, Department of Civil & Structural Engineering, Universiti Kebangsaan Malaysia, UKM Bangi 43600, Selangor, Malaysia, drmrt@vlsi.eng.ukm.my.
[3] Lecturer, Department of Civil Engineering, Rajshahi University of Engineering & Technology, Rajshahi 6204, Bangladesh, dipok47@yahoo.com.

result in the reduction of the cost of construction. However, the interaction mechanism of the reinforced residual soil and the mobilization of the tensile strain in the reinforced composites are not yet well understood due to limited study. A limited amount of information is presently available on studies related to reinforced clay with plastic reinforcement under different drained conditions (Ingold, 1983; Ingold and Miller, 1983). Several successive geosynthetic applications have presented the potential of using c-ϕ type soils as backfill materials (Delmas et al., 1992; Tatsuoka et al., 1996). As a result of the growing interest in utilizing such soils in reinforced soil structures, research on the subject of the geosynthetic-cohesive soil interface behaviour has been intensified (Athanasopoulos, 1996). For the reasons mentioned above, a thorough study of the soil reinforcement interaction on a residual soil was conducted. The study reported in this paper presents the stress-deformation and contraction-dilation characteristics of a reinforced residual soil and the determination of model parameters for the prediction of the behaviour of the system in direct shear test. An elasto-plastic model assuming strain-hardening behaviour (load transfer model) was used to model its behaviour. The experimental test results were adopted to obtain the model constants. The shear stress-strain and volume change properties of the reinforced residual soil composites were analysed and the measured values were compared to that of the model.

SOIL AND REINFORCEMENT PROPERTIES

The soil samples used in this study were obtained from a site in the Cheras, Selangor Malaysia. The decomposed granite is reddish in colour. The soil used in this work is classified as CH in the Unified Soil Classification System (USCS) having specific gravity G_s = 2.63, liquid limit LL= 73% and plastic limit PL = 39%. It contains 46 % clay, 18 % silt, 36 % sand and no gravels. The maximum dry density, γ_d = 14.42 kN/m^3 and optimum moisture content, w_{opt} = 24.6 % were found from the standard compaction test. In this study, a non-woven geotextile was used as the reinforcement material. The tensile strength properties of the reinforcement were determined following ASTM D4595-1992 (ASTM 1992) and can be expressed as

$$\sigma_1 = \frac{P_b}{W} \tag{1}$$

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 physical and mechanical stress-strain properties of the reinforcement are summarised in Table 1.

TABLE 1. Physical and mechanical properties of the non-woven geotextile

Types of geosynthetic	Colour	Material type	Material properties	Value
Non-woven (Polyfelt TS 60)	Grey	Poly-propylene	Thickness at 2 kPa	2.25 mm
			Mass	245 g/m^2
			Tensile strength (machine direction)	18.68 kN/m
			Tensile strength (cross direction)	19.12 kN/m
			Elongation at maximum load MD*	74 %
			Elongation at maximum load CD**	51 %
*MD = machine direction, **CD = cross direction				

SAMPLE PREPARATION AND TESTING PROGRAM

The soil was first dried under laboratory air-dried conditions then ground and passed through a 2 mm sieve. The dry powder was carefully wetted with a spray gun to the standard optimum moisture content. The moist soil was then stored in sealed plastic bags in a humidity room for about two days before use. The moist residual soil was then compacted in a 300 mm x 300 mm shear box mould by machine compaction to the desired height and unit weight at the optimum moisture content. In the case of reinforced soil, the reinforcement consisted of 300 mm x 200 mm size fabric that was cut from the sheet and placed inside the soil in different orientations.

Four series of tests were carried out on both the unreinforced and reinforced residual soil at normal pressures varying between 50-400 kPa and the strain rate of 0.25 mm/min. This rate is in the range recommended in the research manual (Ingold, 1994). For the residual soil used in the tests, this rate of shearing is taken to represent an undrained loading condition. All the tests were run following immediately the placement and compaction of soil in the shear box which represent mainly the short-term conditions developed in the corresponding field application. The tests were conducted on an ELE computer controlled shear box equipment using different load and deformation transducers. The experimental set up and data-logging system is shown in Fig.1.

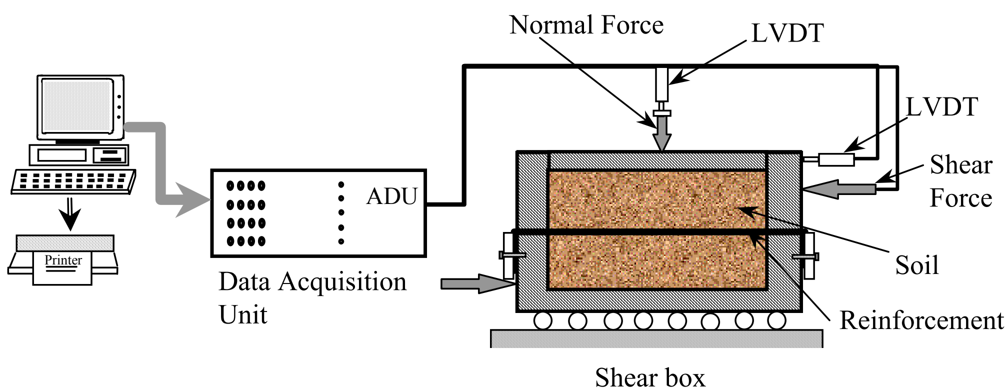

FIG.1. Computer controlled experimental setup of the direct shear test (after Mofiz, 2000)

ELASTO-PLASTIC MODEL

Several constitutive models for normal stress and relative displacement relationship have been developed. Juran et al. (1988) proposed a soil-reinforcement load transfer model assuming an elasto-plastic strain hardening behaviour for sand and an elastic-perfectly plastic behaviour of the reinforcement. Their study allows an evaluation of the effect of the various parameters such as mechanical characteristics and dilatancy properties of the soil, extensibility of the reinforcements and their inclination with respect to the failure surface. In this elasto-plastic model, the soil is considered homogeneous, isotropic, and possesses a strain hardening behaviour. In the analysis of direct shear test results, it is convenient to consider the shear strain as a strain hardening parameter $\gamma_{xy} = \gamma$. The model proposed by Juran et al. (1988) assumes a cohesionless soil. In this study the concept was extended to incorporate cohesive soil which can be readily applied to residual soils.

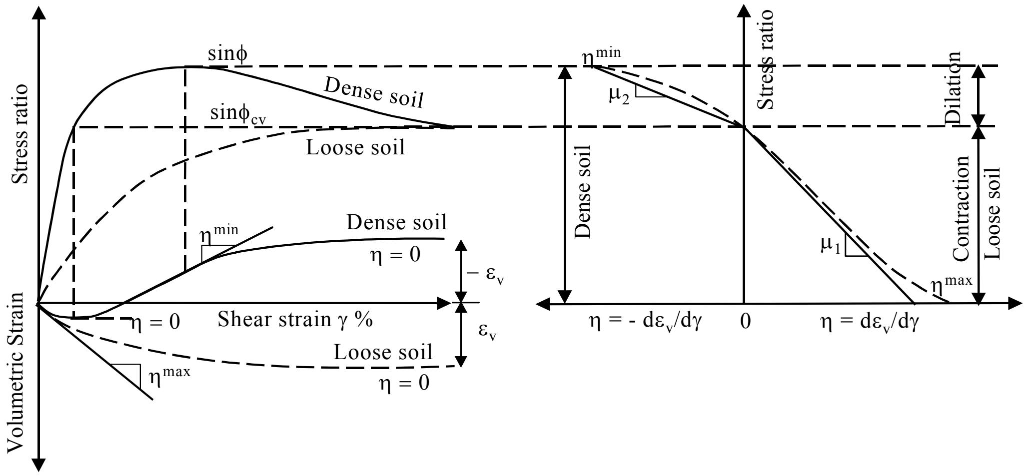

FIG.2. Schematic diagram of stress-strain and volumetric strain characteristics of load transfer model

The yield function is defined by using the Mohr-Coulomb type yield function and can be expressed as

$$f\left(\sigma_{ij}, c, \gamma_{xy}\right) = \frac{\tau_{xy}}{\sigma_y} - \frac{c}{\sigma_y} - S(\gamma_{xy}) = 0 \qquad (2)$$

where τ_{xy} is the shear stress, σ_y is the applied normal stress, c is the cohesion intercept, and the $S(\gamma_{xy})$ function is the related to the $h(\gamma)$ function which is a hyperbolic function. The schematic diagram of the strain hardening function and volumetric strain for both loose and dense soil is shown in Fig.2. The stress ratio-dilatancy rate for both contracting and dilating soils are also shown in the figure. The maximum plastic dilatancy rate, $\eta_{min} = min(d\varepsilon_v/d\gamma)$ is approximately equal to the slope of the volumetric strain-shear curve at the peak of the stress-strain curve, whereas the dilatancy rate at the residual (critical) state is equal to $\eta = 0$. This soil model needs five parameters i.e., G/σ_n which is the ratio of the initial shear modulus to the normal stress, ϕ is the friction angle, ϕ_c is the critical state friction angle of the soil, μ_1 and μ_2 are the contracting and dilating correction factors which is defined in Fig.2. All these parameters may be determined from the analysis of either direct shear test or conventional triaxial test results. It should be noted that these soil properties are functions of the applied normal stress at the interface, soil density and moisture content. Therefore the average characteristics of the compacted residual soil to a given maximum standard Proctor density were used to represent its behaviour in each specified range of normal stresses.

For loose soil, the strain hardening or softening function $h(\gamma)$ is a hyperbolic function and can be expressed as

$$h(\gamma) = \frac{\gamma}{(a + b\gamma)} \qquad (3)$$

The two constants a and b can be evaluated from the experimental data of the direct shear test in which $a = \sigma_n/G$, and $b = 1/tan\phi_{cv}$. For the case of medium to high density soils, it is assumed that the hardening function $h(\gamma)$ is a parabolic function and can be written as

$$h(\gamma)=\frac{F\gamma(\gamma-a)}{(\gamma+b)^2} \tag{4}$$

where the constants F, a and b are determined from the following equations for the direct shear test: $F = \tan\phi_{cv}$, $a = -4(\sigma_n/G)(\tan^2\phi\, l^2)/\tan\phi_{cv}$ and $b = 2(\sigma_n/G)(\tan\phi\, l)$ with $l=1+[1-\{\tan\phi_{cv}/\tan\phi)\}^{1/2}$.

RESULTS AND DISCUSSION

The test results of unreinforced soil with different interfacial normal stresses are shown in Fig 3. The shear stress-shear displacement plot indicates that the relative shear displacement corresponding to maximum shear stress increases with interface normal stress. Fig. 4 shows the shear stress versus horizontal displacement behaviour for reinforced soil with different interface normal stress. As expected, the reinforced soil samples exhibit higher shear strength than unreinforced samples and the maximum shear strength were attained at higher shear strains. The shear stress of unreinforced specimen was reduced after the post peak value. On the other hand for reinforced soil, strain-hardening behaviour was observed due to conversion of brittle for the unreinforced soil to ductile behaviour of the composite material. The shear stress and shear displacement of all unreinforced and reinforced samples at the initial shearing were similar since the effect of the reinforcement will only begin to function at some finite shear displacement. The result shows different strain pattern at higher displacement when the soil samples started to dilate. From the test results it is also observed that the vertical displacement vs. shear displacement behaviour revealed that expansion is more pronounced especially at lower normal stress. The results of unreinforced and reinforced soil samples also show that dilatancy is dependent on the normal stress. The best fit straight line failure envelope (Fig.5) indicates a cohesive-frictional behaviour with strength parameters in terms of total stresses for unreinforced soil in which ϕ = 29.03^o, c = 100.6 kPa and for reinforced soil: ϕ_r = 32.95^o, c = 118.46 kPa, respectively.

The results of the shear stress versus shear displacement and dilation versus shear displacement with reinforcement orientation at 0^o, 45^o and 90^o are presented in Fig.6. It is observed that the shear strength increases with reinforcement orientation, and it was more effective when the orientation was 45^o to the shear plane. This means that the failure stress increases due to the normal and tangential components of the reinforcement. At 45^o orientation the combined effects of the shear displacement and soil dilatancy will mobilize the most tension force in the reinforcement. A similar observation was made by Fantani et al. (1991), Jewell and Wroth (1987) and Gray and Ohashi (1983). The initial volume change properties for both the unreinforced and reinforced soil were contractive and similar. At higher strains, the reinforced soil exhibited greater dilation than unreinforced soil. In such case the reinforced composites behaved as ductile failure behaviour and dilative volume change which causes an increase in the shear strength of the soil composites. A comparison between unreinforced and reinforced soil with respect to the peak shear stress versus normal interface stress with different reinforcement orientations can be deduced from Fig.7. From this figure it is observed that the failure envelope of the unreinforced soil exhibit linear behaviour whereas a curvilinear or bilinear behaviour is illustrated for reinforced soil. This behavior is in agreement with the results published by other researchers (eg. Ranjan et al., 1996 and Athanasopoulos, 1996). The model constants obtained from plots of direct shear test results for unreinforced soil are G/σ_n = 85.68, c = 100.6 kPa, ϕ = 29.03^o, ϕ_{cv} = 28.18^o, μ_1 = 2.46, and μ_2 = 3.48. The reinforced soil constants are G/σ_n = 72.65, c = 118.46, ϕ = 32,95^o, ϕ_{cv} = 30.62^o, μ_1 = 3.16 and μ_2 = 4.10, respectively.

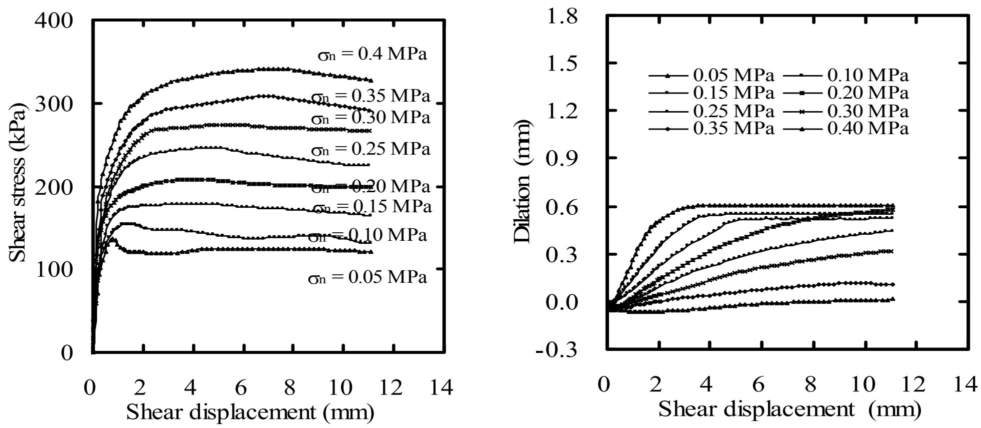

FIG.3. Stress vs. shear deformation and dilation vs. shear deformation of unreinforced soil

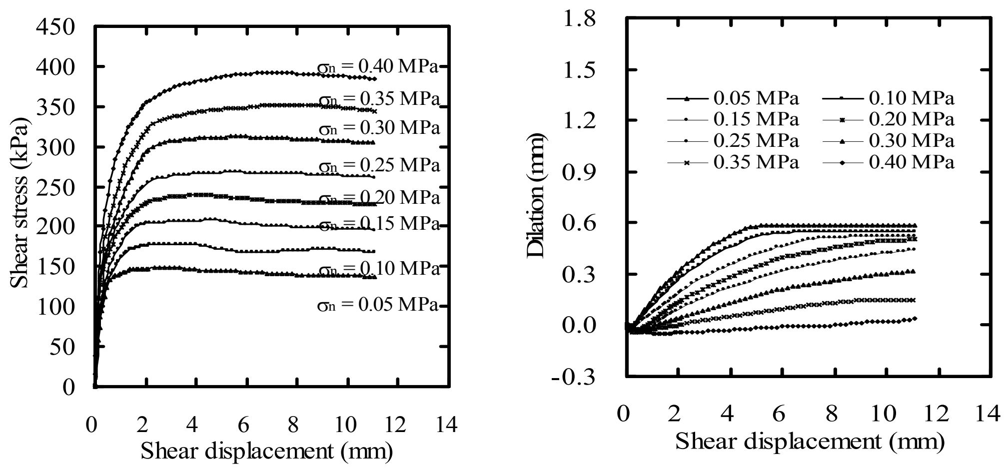

FIG.4. Stress vs. displacement and dilation vs. displacement of reinforced soil composite

The predicted responses of the load transfer model (elasto-plastic model) and measured shear stress-displacement for unreinforced and reinforced soil is shown in (Fig.8, Fig.9 and Fig.10). The general trends of the curves are similar. However, the stress-strain responses for reinforced soil are better than that of unreinforced soil especially for higher normal pressures. Thus, the model prediction can be used to estimate the behaviour reinforced residual soil at these pressures. The trend of predicted curve for different interface normal stress values is in fare agreement with the test data at small displacements. The prediction is also good for higher normal stresses compared to cases at lower normal stresses since the unreinforced and reinforced soil shows a distinct strain softening behaviour. The model predicts (as it assumed) a strain-hardening phenomenon throughout the tests which is not shown to be the case of the test results at lower confining pressures. Thus, prediction using the load transfer model may be used to simulate the dilation-contraction behaviour of the unreinforced and reinforced soil at higher normal pressures. It is evident that a strain-softening model is required for cases at lower confining pressures.

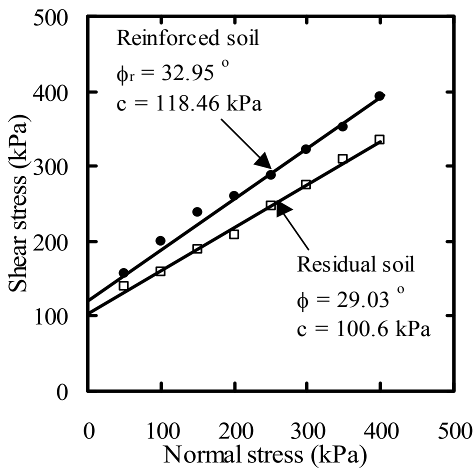

FIG.5. Best fit failure envelope of unreinforced and reinforced soil

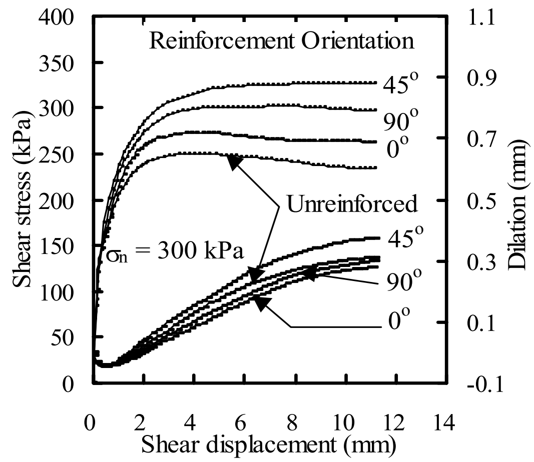

FIG.6. Stress-displacement behaviour of reinforced with different orientation

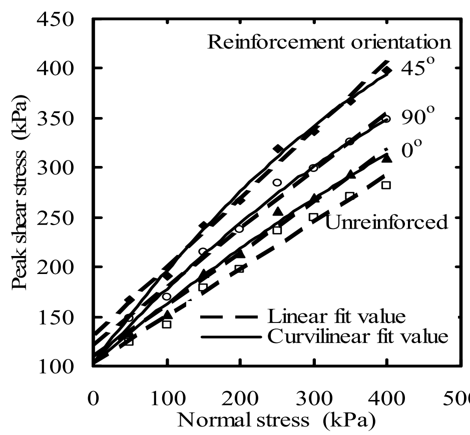

FIg.7. Comparison of failure envelopes of the unreinforced and reinforced soil

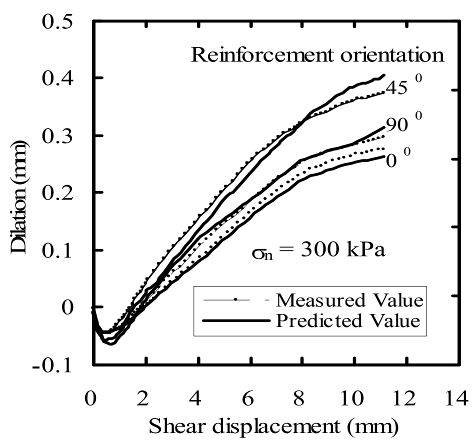

FIG.8. Measured and predicted responses of dilation versus shear displacement

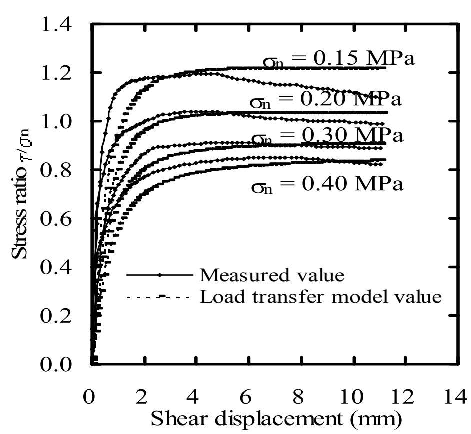

FIG.9. Measured and predicted responses of the unreinforced soil

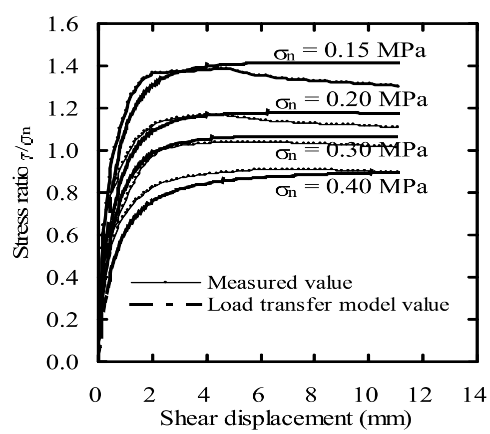

FIG.10. Measured and predicted responses of the reinforced soil

CONCLUSIONS

In order to examine the stress-strain characteristics of unreinforced and reinforced granite residual soil a testing program was carried out in a modified direct shear apparatus. The load transfer model of Juran et al. (1988) was extended for the case of cohesive soil and was tested for its validity for the residual soil. Test results showed that the reinforcement inclusion significantly increases the ultimate shear strength. The composite soils system also fails at relatively larger shear displacement and in most of the cases the reinforced soil shows a strain hardening behaviour. It was observed that the shear strength increases with the reinforcement orientation, and it was more effective when the orientation was 45^o to the shear plane. Failure of the composite soil system may be of two different patterns indicated by a linear or bilinear/curvilinear failure envelope. Prediction using the model constants provides good agreement with the experimental results particularly for reinforced soil at high normal stresses.

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