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Finite element analysis of a reinforced soil

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ABSTRACT: Reinforced soil is used in various geotechnical engineering structures such as retaining walls, foundations, embankments and pavements. Stress-strain and volume change behaviour thus depends on the stress path followed in the field conditions. Due to different loading conditions, it is essential to develop appropriate constitutive model to predict the behaviour of reinforced soil based on laboratory or field tests.

In the present paper, finite element analysis of a non-woven geotextile reinforced soil is presented. The analysis has been made using hierarchical model developed by Desai and his co-workers based on the theory of elasto-viscoplasticity. Verification of the model has been made with the laboratory triaxial tests conducted on geotextile reinforced Ennore sand using various stress paths.

1 INTRODUCTION

Reinforced soil is used in various structures including retaining walls, foundations, embankments and pavements. Due to wide applications, reinforced soil is subjected to different loading conditions. Since the field tests are very costly, it is essential to develop suitable constitutive model to predict the behaviour of reinforced soil based on the laboratory tests using various stress paths.

The characterisation of such a reinforced soil has been made using to characterise the behaviour of the soil and the interface and von-Mises criterion for the reinforcement.

To validate the model, drained triaxial tests are conducted using six different stress paths on unreinforced and reinforced soils. Modified direct shear tests are conducted to obtain the interface properties. Predicted results of stress-strain-volume change response are compared with the experimental results.

2 LABORATORY TESTS

2.1 *Soil*

Ennore sand commonly known as Indian Standard sand of specific gravity 2.64; uniformity coefficient 1.63; effective size, D_{10}, 0.40 mm and median size, D_{50}, 0.60 mm is used. Maximum and minimum dry unit weights of the sand are 18 kN/m^3 and 16 kN/m^3 respectively.

2.2 *Reinforcement*

White coloured non-woven geotextiles made of polypropylene is used as the reinforcement. The thickness of the geotextile is 2.80 mm at 2 kPa, the average tensile strength is 11.65 kN/m and the elastic modulus is 23.13 kN/m.

2.3 *Experimental programme and set up*

Drained triaxial tests have been conducted on cylindrical samples of approximately 38 mm in diameter and 76 mm in height at the relative density of 65 percent. The reinforcement discs have been placed horizontally at mid height of the sample. The samples have been subjected to failure along six different stress paths as given in Fig.1 in which σ_1

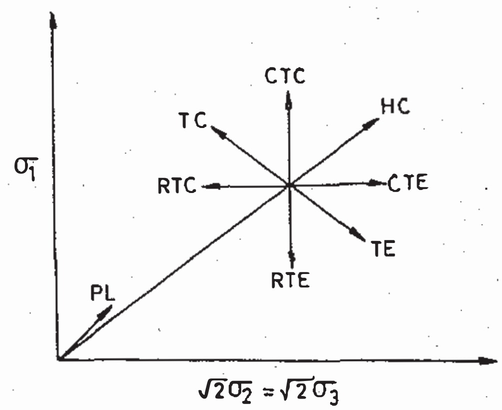

Fig.1 Schematic diagram of stress paths

represents the axial stress and σ_2 (= σ_3), the radial stress.

The tests have been conducted on GDS computer controlled triaxial equipment using digital pressure controllers to apply cell pressure, axial stress and to measure volume change.

2.4 *Experimental procedure*

Cylindrical unreinforced samples were prepared as per the standard procedure using the sample former (Bishop and Henkel, 1957). For reinforced soil, preweighed sample was taken into two equal parts. After placing the former as in case of unreinforced sample, first part of the soil was poured. The compaction was done through tamping procedure. The reinforcement of 38 mm diameter was then placed over the soil. Second part of the sample was then poured and compaction carried out. Suitable top cap was used according to the compression or extension path to be followed.

Modified direct shear tests on a box size of 6 cms x 6 cms x 2 cms (Rao and Pandey, 1988) were conducted to find out the interface properties. The geotextile was wrapped over the dummy block and upper portion was allowed to slide over the dummy block.

3 FINITE ELEMENT ANALYSIS

Considering the symmetry, only one half part of the reinforced soil sample is discretised into ten 8-noded isoparametric (Hinten and Owen, 1977) solid elements and four 6-noded interface elements having 55 nodes. The soil and the reinforcement are modelled as 8-noded solid elements. Axisymmetric condition is assumed. In-situ stresses have been kept equal to initial consolidation stresses. Uniform pressure loading is then applied according to the stress path followed.

The equilibrium equation for total assembly in terms of global stiffness matrix [K] is

$$[K]\{\delta\} = \sum_{e=1}^{x} \int_V [B]^T [D] \{\epsilon^{vp}\} dV + \{R\}$$

where $\{\delta\}$ is the vector of unknown global displacements, $\{R\}$ is the global load vector obtained by the summation of forces on the nodes due to the contribution from all the elements to which this node is connected and x is the total number of elements. [D] is the elasticity matrix, [B] is the strain displacement matrix and V is the volume. ϵ^{vp} is the viscoplastic strain. Elasto-viscoplasticity is used as an artifice to obtain the elasto-plastic solutions.

A convergence limit equal to 0.01 is applied for elasto-viscoplastic algorithm to avoid abnormally large number of steps to achieve cent percent convergence. To calculate incremental viscoplastic strains from the viscoplastic strain rates, variable time step length is adopted (Zienkiewicz and Cormeau, 1974). Total time steps used in the analysis are kept as 400. Time increment factor is kept as 0.05.

3.1 *Constitutive model*

The continuous yield behaviour is given by a compact and specialised form (Desai, 1980; Desai et al., 1986).

$$F = \frac{J_{2D}}{p_a^2} - \left[-\alpha \left(\frac{J_1}{p_a}\right)^n + \gamma \left(\frac{J_1}{p_a}\right)^2 \right] (1 - \beta S_r)^m = 0 \qquad (1)$$

$$\text{or} \quad F = \frac{J_{2D}}{p_a^2} - F_b F_s = 0 \qquad (2)$$

where J_{2D} is the second invariant of the deviatoric stress tensor; J_1 is the first invariant of the stress tensor; p_a is the atmospheric pressure; γ, β, m are the material response functions associated with ultimate behaviour in which m is taken as -0.5, α is the hardening function; n is the phase change parameter and S_r is the stress ratio given by

$$S_r = \frac{\sqrt{27}}{2} \frac{J_{3D}}{J_{2D}^{1.5}} \qquad (3)$$

in which J_{3D} is the third invariant of deviatoric stress tensor. In Eq.(2), F_b is the basic function describing the shape of the yield function in $J_1 - \sqrt{J_{2D}}$ space, F_s is the shape function describing the shape in the octahedral plane.

The hardening function α is given by (Frantziskonis et al., 1986) as $\alpha = a_1/\xi^{\eta_1}$ in which a_1 and η_1 are the hardening parameters and $\xi = \int (d\epsilon_{ij}^p \, d\epsilon_{ij}^p)^{1/2}$ the trajectory of the plastic strain. ξ can be splitted into volumetric (ξ_V) and deviatoric (ξ_D) components.

Non-associative flow rule is applied using a potential function, Q defined as (Desai and Hashmi, 1989).

$$Q = \frac{J_{2D}}{p_a^2} - \left[-\alpha_Q \left(\frac{J_1}{p_a}\right)^n + \gamma \left(\frac{J_1}{p_a}\right)^2 \right] F_s \qquad (4)$$

where growth function α is replaced by α_Q and is equal to (Soni, 1995)

$$\alpha_Q = \alpha + \kappa \frac{J_1 \, p_a}{J_{2D}} (\alpha_o - \alpha)(1 - r_v) \quad (5)$$

where α_o is the value of α at the end of initial (hydrostatic) loading and κ is non-associative parameter defined as

$$\kappa = \frac{J_{2D}}{J_1 \cdot p_a} \left[(\alpha_Q - \alpha)/(\alpha_o - \alpha)(1 - r_v) \right] \quad (6)$$

Procedure to find out the material parameters is described by (Desai et al., 1986; Soni, 1995 among others).

Yield criterion for the interface can be written in normalised form as (Desai and Fishman, 1991).

$$F = \left[\frac{\tau}{p_a}\right]^2 + \alpha \left[\frac{\sigma_n}{p_a}\right]^n - \gamma \left[\frac{\sigma_n}{p_a}\right]^2 = 0 \quad (7)$$

where τ is the shear stress; σ_n, the normal stress; γ, the ultimate parameter and α, the hardening parameter.

Non-associativeness is introduced for interface behaviour in a similar way to the solid elements as,

$$Q = \left[\frac{\tau}{p_a}\right]^2 + \alpha_Q \left[\frac{\sigma_n}{p_a}\right]^n - \gamma \left[\frac{\sigma_n}{p_a}\right]^2 = 0 \quad (8)$$

in which $\alpha_Q = \alpha + \kappa (\alpha_o - \alpha)(1 - r_v)$

3.2 *Computer programs*

A computer programs PARAMV (Soni, 1995) has been used to determine the material constants for the soil summarised in Table 1. Another program PARAMJ (Soni, 1995) has been used for determining the material constants of the interface presented in Table 2. K and n' are the elastic constants of Janbu's relation. K_s and K_n are the shear and normal stiffness of the interface. For prediction of stress-strain-volume change response of reinforced soils, another computer program VISCPL (Soni, 1995) has been used.

Table 1. Material constants of unreinforced soil

Elastic constants	K n' ν	600 0.95 0.34
Ultimate parameters	m γ β	-0.5 0.071 0.610
Phase change parameter	n	2.54
Hardening parameters	a_1 η_1	0.366×10^{-3} 0.711
Non-associative parameter	κ	0.228

Table 2. Material constants of interface behaviour

Elastic constants (kN/m²/m)	K_s K_n	3000 60000
Ultimate parameter	γ	0.722
Phase change parameter	n	2.84
Hardening parameters	a_1 η_1	0.098 0.675
Non-associative parameter	κ	0.875

4 PREDICTIONS

Predictions of the stress-strain-volume change response of typical tests are shown in Figs. 2 to 4. It is observed that the predictions are satisfactory compared to the observed response, showing the validation of the model used.

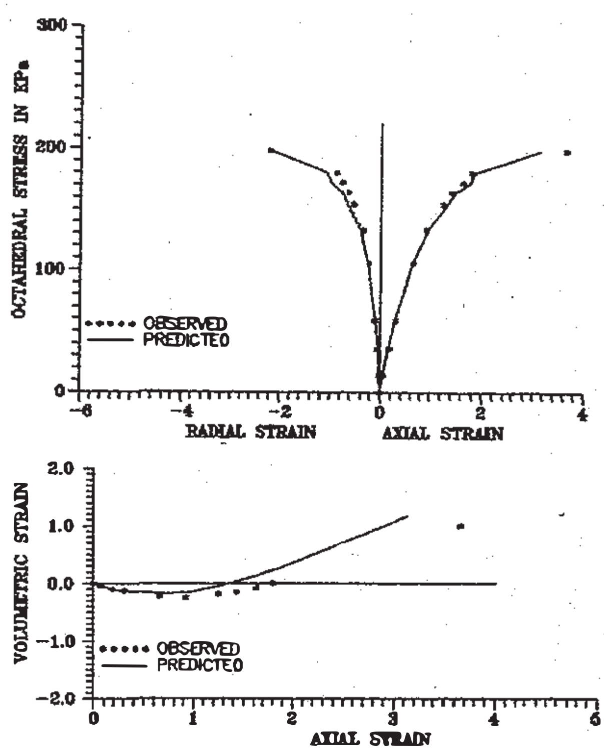

Fig.2 Stress-strain-volume change response for CTC path at σ_c = 100 kPa

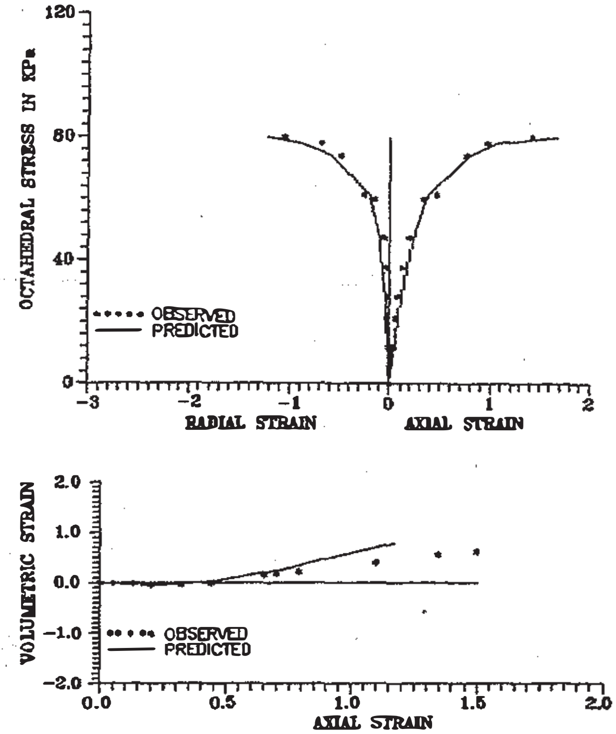

Fig.3 Stress-strain-volume change response for TC path at σ_c = 100 kPa

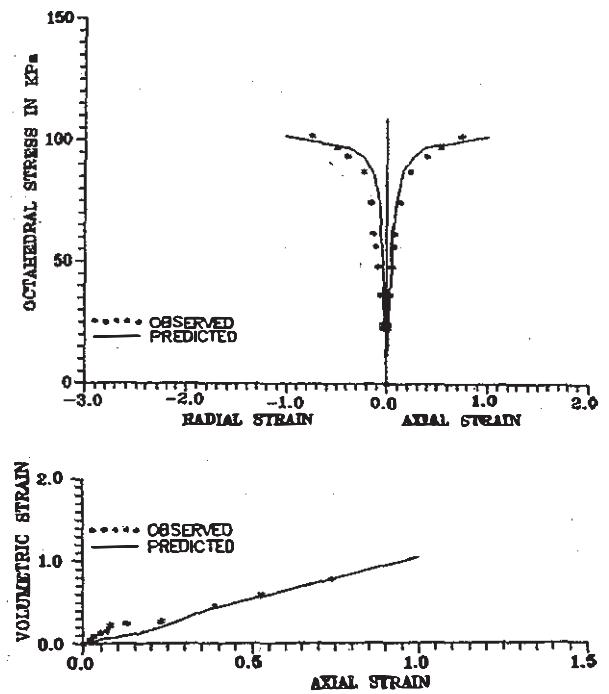

Fig.4 Stress-strain-volume change response for RTC path at σ_c = 300 kPa

5 CONCLUSIONS

Finite element analysis is presented to predict the stress-strain and volume change behaviour of reinforced soils using hierarchical model for the soil and the interface and von-Mises criterion for the reinforcement. Satisfactory predictions of the observed stress-strain-volume change response show the validity of the model. The model can therefore be implemented in solution procedures for field problems.

REFERENCES

Bishop, A.W. and Henkel, D.J. 1957. The measurements of soil properties in the triaxial test. Edward Arnold Ltd., London.

Desai, C.S. (1980). A general basis for yield, failure and potential functions in plasticity. Int. Jl. Num. Anal. Methods in Geomechanics, 4:361-375.

Desai, C.S. and Fishman, K.L. (1991). Plasticity based constitutive model with associative testing for joints. Int. Jl. RMMS 28(7): 15-26.

Desai, C.S. and Hashmi, Q.S.E. (1989). Analysis, evaluation and implementation of a non-associative model for geological materials. Int. Jl. of Plasticity, 5:397-420.

Desai, C.S., Somasundaram, S., Frantziskonis, G.N. (1986). A hierarchical approach to constitutive modelling of geological materials. Int. Jl. Num. Anal. Methods in Geomechanics, 10(3):225-257.

Frantziskonis, G., Desai, C.S. and Somasundaram, S. (1986). Constitutive model for non-associative behaviour. Jl. of Engg. Mech. of ASCE, 9:932-946.

Hinten, E. and Owen, D.R.J. (1977). An introduction to finite element computations. Pineridge Press, Swansea, U.K.

Rao, G.V. and Pandey, S.K. (1988). Evaluation of geotextile soil friction. Indian Geot. Jl., 18(1): 77-105.

Soni, K.M. (1995). Constitutive modelling of reinforced soil. Ph.D. Thesis, Civil Engg. Dept., I.I.T. Delhi, India.

Zienkiewicz, O.C. and Cormeau, I.C. (1974). Viscoplasticity, plasticity and creep in elastic solids - a uniform numerical solution approach. Int. Jl. Num. Methods in Engg., 8:821-845.