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MODELLING MEMBRANE EFFECT ON PORE PRESSURES IN TRIAXIAL TESTS

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ABSTRACT

Cyclic triaxial tests are most commonly used in the liquefaction analysis of saturated sand deposits. Since this analysis is based on the effective stress principle, its accuracy depends upon the precision with which the pore-water pressure measurement is made. In the conventional triaxial tests the pore pressure generation is influenced by various test factors of which the membrane penetration is the major source of error. Its effect is to impede the pore pressure development owing to its penetration into the peripheral voids of a granular sample producing volume changes in it under varying effective confining pressure. This evidently invalidates the no-volume-change condition of an undrained test. From the results reported in literature the pore pressure development in a conventional test is about 75 to 100% lower in loose samples due to membrane effect. Therefore, there is great need to eliminate its effect experimentally as this phenomenon is not associated with field behaviour. Efforts of the writer and other investigators to experimentally eliminate this effect have met with partial success only. This has, in fact, prompted the use of alternative analytical methods to simulate the effect of membrane penetration and assess the true pore pressure generation. In this endeavour two analytical models are presented to predict the true pore pressures fully accounting for the membrane effect. These models employ the membrane compliance ratio, which is the ratio of the coefficient of membrane penetration and the rebound volume modulus of sand skeleton. These parameters are obtained from the membrane penetration tests and the first rebound curve of drained isotropic compression tests respectively. The results of application of these models to predict the true pore pressures in static and cyclic undrained triaxial tests have clearly demonstrated their effectiveness as an alternative method.

1.0 INTRODUCTION

The implicit assumption in an undrained triaxial test is that the volume changes occurring during the test are zero. However, it is well established that the membrane confining the soil sample penetrates into the peripheral voids of the sample as the effective confining pressure changes during the test and causes volume changes. This invalidates the above assumption. The effect of membrane penetration is to greatly reduce the rate of change in the pore-water pressure (4,8). The mechanism of membrane penetration is illustrated in Figure 1. As the membrane penetration is a function of grain size (2,9) its effect will be significant in the case of granular samples. Its elimination is inevitable in order that the laboratory assessment of strength of sand deposits and their liquefaction potential becomes

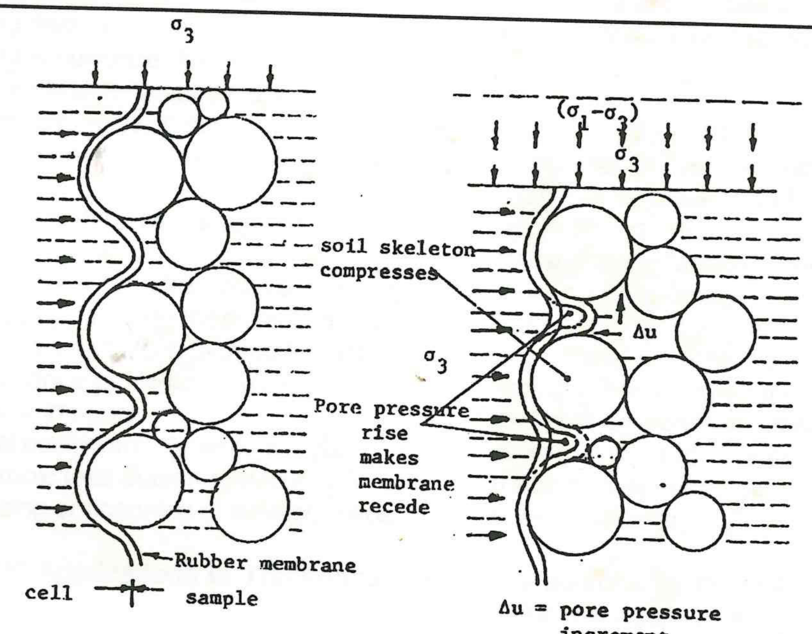

Figure 1 Schematic showing Membrane Effect on Pore Pressure Generation

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more reliable for field use. Many attempts to eliminate this effect of membrane penetration experimentally have met with partial success only (3,7). Constant volume tests for complete elimination of this effect described by the writer (8) is difficult to apply as routine tests. Obviously there is need to develop analytical models to simulate the membrane effect on pore pressure development in triaxial tests.

2.0 ANALYTICAL MODELS

Two analytical models are proposed to compute true pore pressure (corresponding to no membrane effect):

(1) Corrector: which corrects the measured pore pressures
(2) Predictor: which predicts the experimental and true pore pressures

These analytical models are derived using the basic procedure as follows:

Basic Procedure

The fundamental assumption in an undrained triaxial test is that the overall volume change of a saturated sample is zero. This implies that ideally the volume changes that occur in the soil skeleton, pore-water and those due to membrane penetration must all balance so that the net volume change in the sample is zero. Thus, for a saturated sample,

$$\frac{dV_s}{V} + \frac{dV_m}{V} = \frac{dV_w}{V} \qquad (1)$$

where
dV_s = volume change of soil skeleton
dV_m = volume change due to membrane penetration
dV_w = volume change of the pore-water caused by a change in the pore-water pressure
V = initial sample volume

Volumetric strain, dV_s/V that occurs in the soil skeleton may be expressed as,

$$\frac{dV_s}{V} = \frac{dV_c}{V} + \frac{dV_d}{V} \qquad (2)$$

where $\frac{dV_c}{V}$ represents consolidation component

and $\frac{dV_d}{V}$ represents dilatancy component

The consolidation component is due to the mean effective consolidation pressure,

$$dP' = \frac{(d\sigma_1 + 2d\sigma_3')}{3}$$

where $d\sigma'$ and $d\sigma_3'$ are the increments in the effective principal stresses, whereas the dilatancy component is due to the octahedral shear stress, τ_{oct}, which for a conventional triaxial test equals

$$\frac{1}{3}\left[(\sigma_1 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2\right]^{1/2} \qquad (2)$$

Rewriting equation (2) using the above terms,

$$\frac{dV_s}{V} = C_s(dp - du) + \frac{D'}{P_c'}d(\tau_{oct}^2) \qquad (3)$$

where
C_s = rebound volume modulus or the slope of elastic volumetric strain versus mean effective stress curve
dp, du = increments in the mean stress and pore pressure respectively
D' = empirical parameter of volume change under shear
τ_{oct} = octahedral shear stress. Square of τ_{oct} is used as dilatancy is independent of direction of shearing
P_c = initial consolidation pressure. It is used to account for the influence of initial consolidation pressure on volume changes

Substituting for τ_{oct} and simplifying equation (3) reduces to,

$$\frac{dV_s}{V} = C_s(dp - du) + \frac{D}{P_c'}\, d(\sigma_1 - \sigma_3)^2 \qquad (4)$$

where $D = \frac{2}{9}D'$

Volume changes arising out of membrane penetration due to a change in the mean effective confining pressure are given by,

$$\frac{dV_m}{V} = C_m dp' = C_m(dp - du) \qquad (5)$$

C_m represents the coefficent of membrane compliance or the slope of $\frac{dV_m}{V}$ versus dp' curve.

On account of excess pore pressure developed, du, the pore-water undergoes a volume change, under no drainage, which can be obtained from the relation,

$$\frac{dV_w}{V} = n.C_w.du$$

n = porosity of sample
C_w = coefficient of compressibility of pore-water

Substituting equations (4), (5) and (6) in equation (1) and simplifying results in the following,

$$du = \frac{dp(C_s + C_m) + D\dfrac{d(\sigma_1 - \sigma_3)^2}{P_c'}}{(C_s + C_m + nC_w)} \qquad (7)$$

During consolidation of the sample in triaxial test, assuming full saturation of sample and neglecting the compressibility of pore-water, the above equation reduces to,

$$du = dp$$

$$B' = \frac{du}{dp} = 1$$

While the sample is sheared after consolidation,

$$dp = 0 \text{ and}$$

$$du = \frac{D\ d(\sigma_1 - \sigma_3)^2}{(C_s + C_m)P_c'} \qquad (9)$$

This equation gives the pore pressure that develops in a conventional undrained triaxial test where in full membrane penetration is allowed to occur. Rewriting the above equation,

$$A' = \frac{du}{d(\sigma_1 - \sigma_3)} = \frac{D\ d(\sigma_1 - \sigma_3)}{(C_s + C_m)P_c'} \qquad (10)$$

B' and A' are the modified forms of Skempton's pore pressure parameters. In an ideal triaxial testing system, the effect of membrane penetration is zero and hence equation (9) simplified by putting $C_m = 0$, gives the true pore pressure, du_t.

$$du_t = \frac{D\ d(\sigma_1 - \sigma_3)^2}{C_s\ P_c'} \qquad (11)$$

Comparing equations (9) and (11) the relation between the true and measured pore pressures can be obtained.

$$du_t = du\left(1 + \frac{C_m}{C_s}\right) \qquad (12)$$

This equation is called the corrector. If C_m/C_s is replaced by C_R, called the membrane compliance ratio, then

$$du_t = du(1 + C_R) \qquad (13)$$

Substituting for dV_s/V and dV_m/V and neglecting the compressibility of pore-water equation (1) is modified as:

$$\frac{dV_d}{V} + C_s(dp - du) + C_m(dp - du) = 0 \qquad (14)$$

In an undrained triaxial test performed with constant cell pressure, dp = 0, then the above equation simplifies to:

$$du = \frac{\left(\dfrac{dV_d}{V}\right)}{(C_s + C_m)} = \frac{d\varepsilon_{vd}}{(C_s + C_m)} \qquad (15)$$

This equation is called the predictor. $d\,\varepsilon_{vd}$ = Volume changes obtained from drained triaxial tests in which membrane penetration does not vary during the test.

Parameters C_s and C_m are obtained from the first rebound curve from drained isotropic compression tests and membrane penetration tests respectively.

Further, these corrector and predictor equations have to satisfy two boundary conditions of membrane penetration. When the pore pressure rise during shear is small then there is little change in the effective confining pressure and hence in membrane penetration. Similarly, when the pore pressure rise is so significant that resultant effective confining pressure is negligible the same result occurs. At these situations the true and measured pore pressures are nearly the same.

These boundary conditions will be satisfied if the compliance ratio used in the above equations is multiplied by the ratio (α) of the current effective confining pressure (σ_3') to the initial consolidation pressure (σ_{3c}'). In view of this the corrector and predictor equations are modified as follows:

$$\text{Corrector} \quad du_t = du\left(1 + \alpha\frac{C_m}{C_s}\right) \qquad (16)$$

$$\text{Predictor} \quad du = \frac{d\varepsilon_{vd}}{C_s\left(1 + \alpha\dfrac{C_m}{C_s}\right)} \qquad (17)$$

$$\text{where} \quad \alpha = \left(\frac{\sigma_3'}{\sigma_{3c}'}\right)$$

3.0 MEMBRANE COMPLIANCE RATIO

As may be noted from equations (16) and (17) the effect of membrane penetration on pore pressures is controlled by the compliance ratio, C_m/C_s. In order to determine the membrane compliance ratios for different grain sizes, measurement of elastic volume changes and membrane penetration values for the same sand under a range of confining pressures is required.

Dummy rod method is commonly used to determine membrane penetration values. It consists of a series of drained isotropic compression tests with dummy rods of different diameters placed coaxially within the cylindrical samples. With increasing size of dummy rod, the volume changes due to compression of sand skeleton decrease while those due to membrane penetration remain constant. Extrapolation of the results to zero sample volume gives volume change due to membrane penetration for the corresponding confining pressure. Elastic volume changes of the soil skeleton are determined by isotropically loading and unloading the same sand samples and measuring the corresponding volume changes. This process is to be repeated until two consecutive loading and unloading cycles gave constant volume changes. These readings are then corrected for membrane penetration which occurs due to varying cell pressures during the test. In all these tests sand samples of height to diameter ratio of unity with lubricated end platens are used. This data of the author and the limited data available in literature, relating to samples of similar geometry are transformed and shown in Figure 2 and Figure 3. From these curves it is obvious that the elastic volume change is fairly independent of the grain size while it varies somewhat with density, Figure 4. Whereas the volume change due to membrane penetration is a function of sample geometry exhibiting slight variation with density. The derived expressions for volumetric strain in each of the cases corresponding to medium density are,

$$\varepsilon_{vm} = 0.5\frac{D_{50}}{d}\log\left(\frac{\sigma_3'}{\sigma_{3i}'}\right) \tag{18}$$

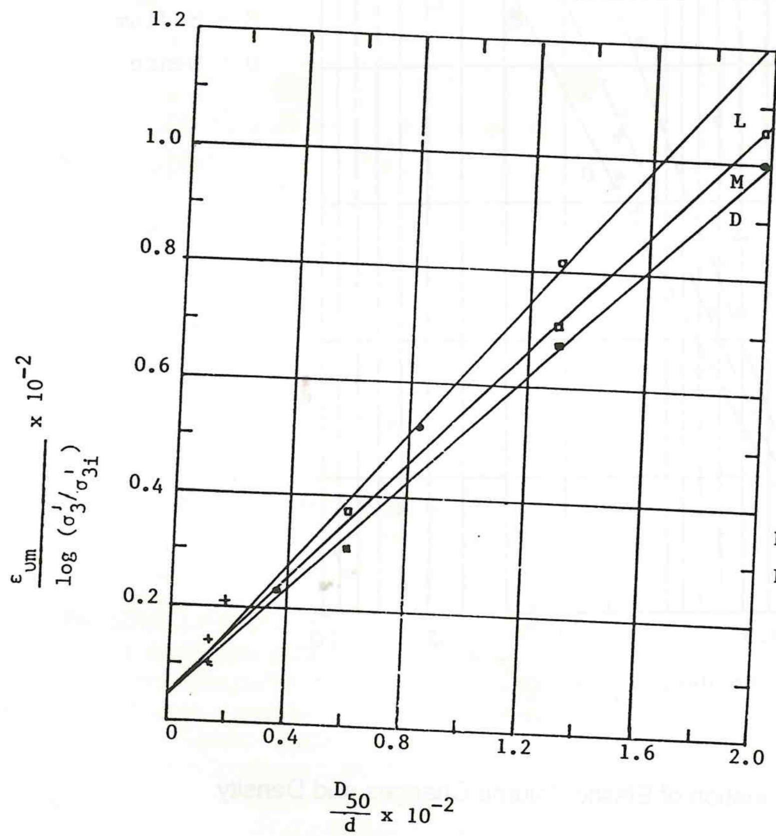
ε_{vm} = volumetric strain due to membrane penetration
D_{50} = mean grain size in mm. It must be noted here that D_{50} is chosen to be the representative grain size. However, R.B. Seed (1989) has observed that D_{20} size correlates better.
d = sample diameter in mm
σ_3' = effective confining pressure
σ_{3i}' = initial consolidation pressure

$$\varepsilon_{ve} = 0.55\ \log\left(\frac{\sigma_3'}{\sigma_{3i}'}\right) \times 10^{-2} \tag{19}$$

ε_{ve} = elastic volumetric strain of sand skeleton

Then the ratio $\varepsilon_{vm}/\varepsilon_{ve}$ is considered as the membrane compliance ratio, C_R. Substituting expressions for ε_{vm} and ε_{ve} from (18) and (19) C_R may be written as:

$$C_R = 90\frac{D_{50}}{d} \tag{20}$$



sand	D_{50} (mm)	d (mm)	Ref.
Fine sand Ennore sand coarse sand	0.28 0.64 1.50	76 76 76	Author
River sand	0.45 0.98 1.50	76 76 76	sadasivan (1974)
Fine sand Mersey River sand	0.13 0.18	100 100	El-sohby (1969)

slope	
L = Loose	0.59
M = Medium	0.50
D = Dense	0.47

Figure 2 Normalised Plots of Variation of Membrane Penetration Values with Grain Size and Density

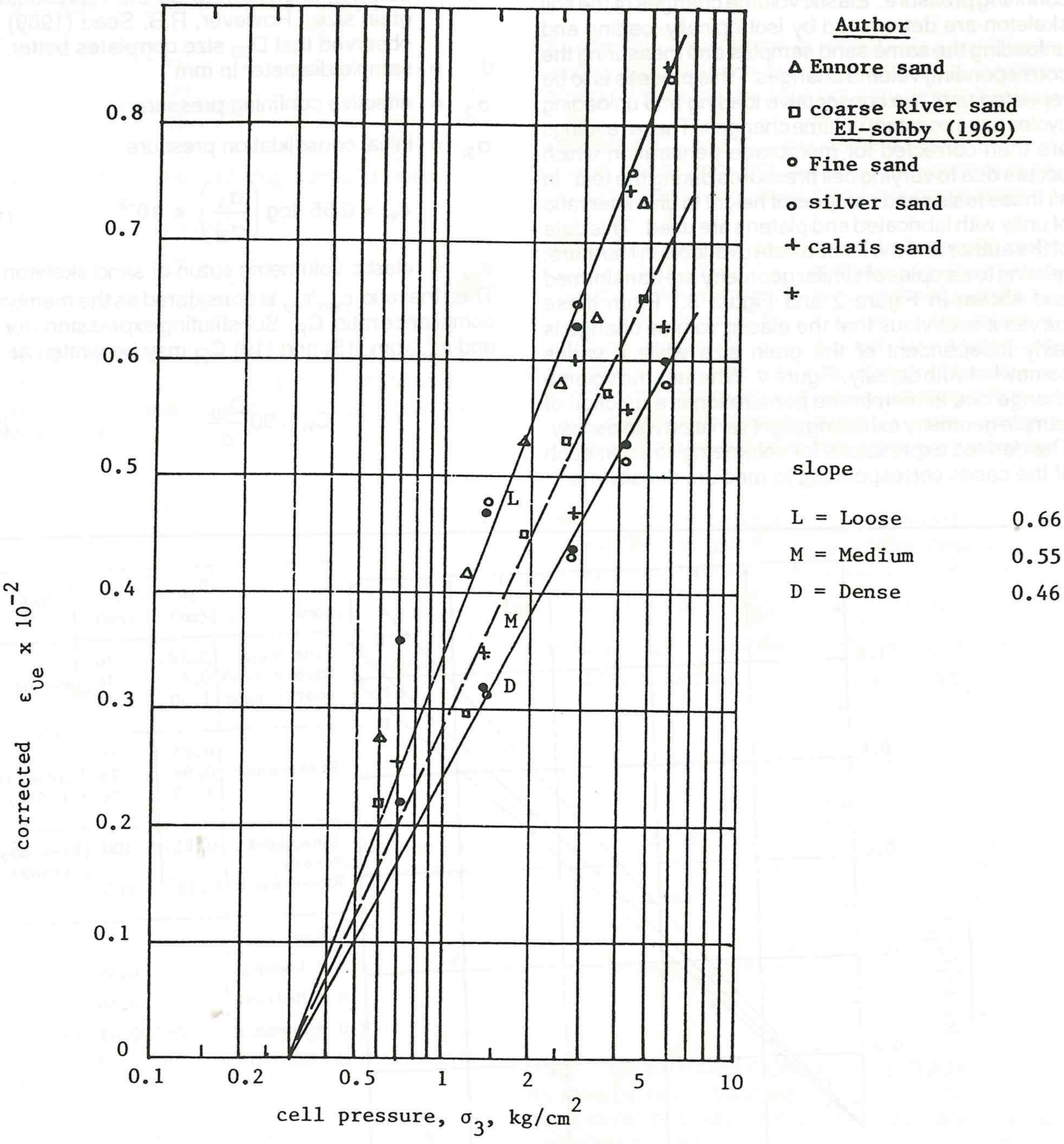


Figure 3 Variation of Elastic Volume Changes and Density

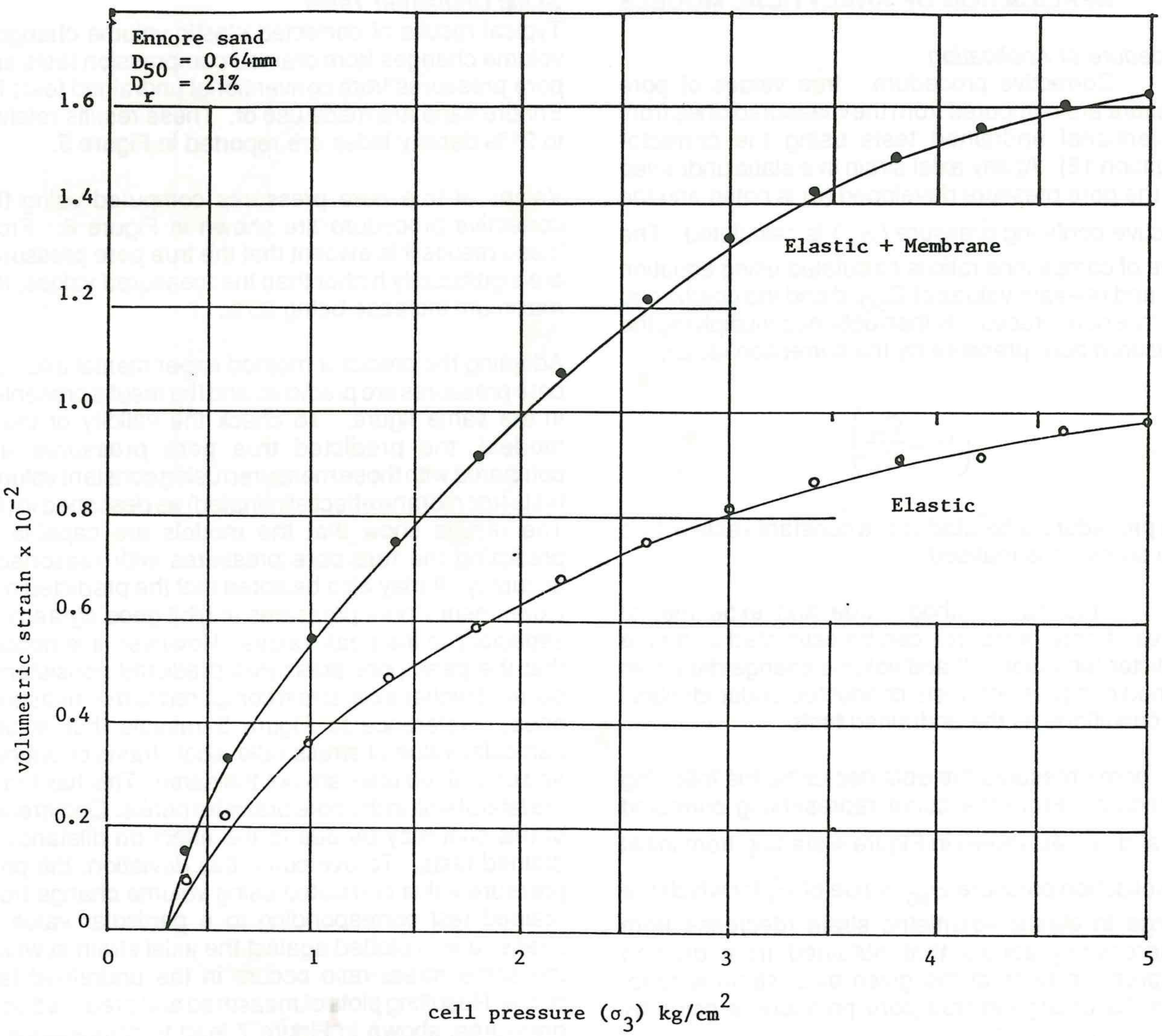


Figure 4 Volume Changes due to Elastic Rebound and Membrane Penetration as a Function of Cell Pressure

The coefficients for loose and dense states of samples obtained from the respective curves of Figure 2 and Figure 3 are 88 and 102 respectively.

4.0 APPLICATION OF ANALYTICAL MODELS

Procedure of Application

(a) Corrective procedure - true values of pore pressure are computed from the measured ones from conventional undrained tests using the corrector (equation 16). At any axial strain in a static undrained test the pore pressure developed (u) is noted and the effective confining pressure (σ_3') is calculated. The value of compliance ratio is calculated using equation (20) and relevant values of D_{50}, d and the coefficient. The true pore pressure is then obtained multiplying the measured pore pressure by the correction factor,

$$\left(1+\alpha\frac{C_m}{C_s}\right)$$

This procedure is iterated until a constant value of true pore pressure is realised.

(b) Predictive method - true and experimental values of pore pressures can be estimated using the predictor (equation 17) and volume change data from drained compression tests conducted under identical test conditions as the undrained tests.

True pore pressures are obtained using the following procedure. From the curve representing corrected ε_{ve} and σ_3', as shown in Figure 4 starting from initial consolidation pressure σ_{3C}', value of σ_3' for which the change in elastic volumetric strain (decrease from compression) equals that obtained from drained compression tests at the given axial strain is read. Then the change in true pore pressure is given by ($\sigma_{3C}' - \sigma_3'$). If such a curve is not available then the appropriate curve of Figure 4 may be used with some loss of precision. If experimental values are to be predicted both elastic volume changes of soil skeleton and volume changes due to membrane penetration have to be considered. First true pore pressure at any axial strain of the static test is computed as described above. This value is then divided by the factor

$$\left(1+\alpha\frac{C_m}{C_s}\right)$$

For the true pore pressure arrived at, α is calculated and hence, the experimental pore pressure. α–value is now revised and a new value of the experimental pore pressure is obtained. This process is iterated until a constant value is obtained.

Results of Application

In order to verify the effectiveness of these models data from static and cyclic undrained tests is applied.

Static Undrained Tests

Typical results of corrected elastic volume changes, volume changes from drained compression tests and pore pressures from conventional undrained tests for Ennore sand are made use of. These results relating to 21% density index are reported in Figure 5.

Values of true pore pressures computed using the corrective procedure are shown in Figure 6. From these results it is evident that the true pore pressures are significantly higher than the measured values, the maximum increase being 65%.

Adopting the predictor method experimental and true pore pressures are predicted and the results presented in the same figure. To check the validity of these models, the predicted true pore pressures are compared with those measured using constant volume tests (membrane effect eliminated) as described in (8). The results show that the models are capable of predicting the true pore pressures with reasonable accuracy. It may also be noted that the predicted and experimental pore pressures exhibit good agreement especially in the peak values. However, it is noticed that the peak pore pressures predicted consistently occur at higher axial strain compared to the measured ones. Reference to Figure 5 reveals that at any particular value of stress ratio axial strains of drained and undrained tests are not the same. This has led to the lateral shift in the pore pressure peaks. Occurrence of this shift may be due to the effect on dilatancy in drained tests. To overcome this deviation, the pore pressure value computed using volume change from drained test corresponding to a particular value of stress-ratio is plotted against the axial strain at which the same stress-ratio occurs in the undrained test curve. Resulting plots of measured and predicted pore pressures, shown in Figure 7 lead to coincidence of peak values.

Cyclic Undrained Tests

Results of pore pressures and number of cycles from one-way and two-way cyclic tests of conventional type in which full effect of membrane penetration occurs are reported in Figures 8 and 9 along with true pore pressures computed using the corrector model. Owing to lack of data on measured volume changes in cyclic tests for sands used in this investigation the predictor model could not be applied.

5.0 CONCLUSIONS

An attempt is made to correct the measured pore pressures for the effect of membrane penetration and also to predict the experimental and true pore pressures in undrained triaxial tests using the two analytical

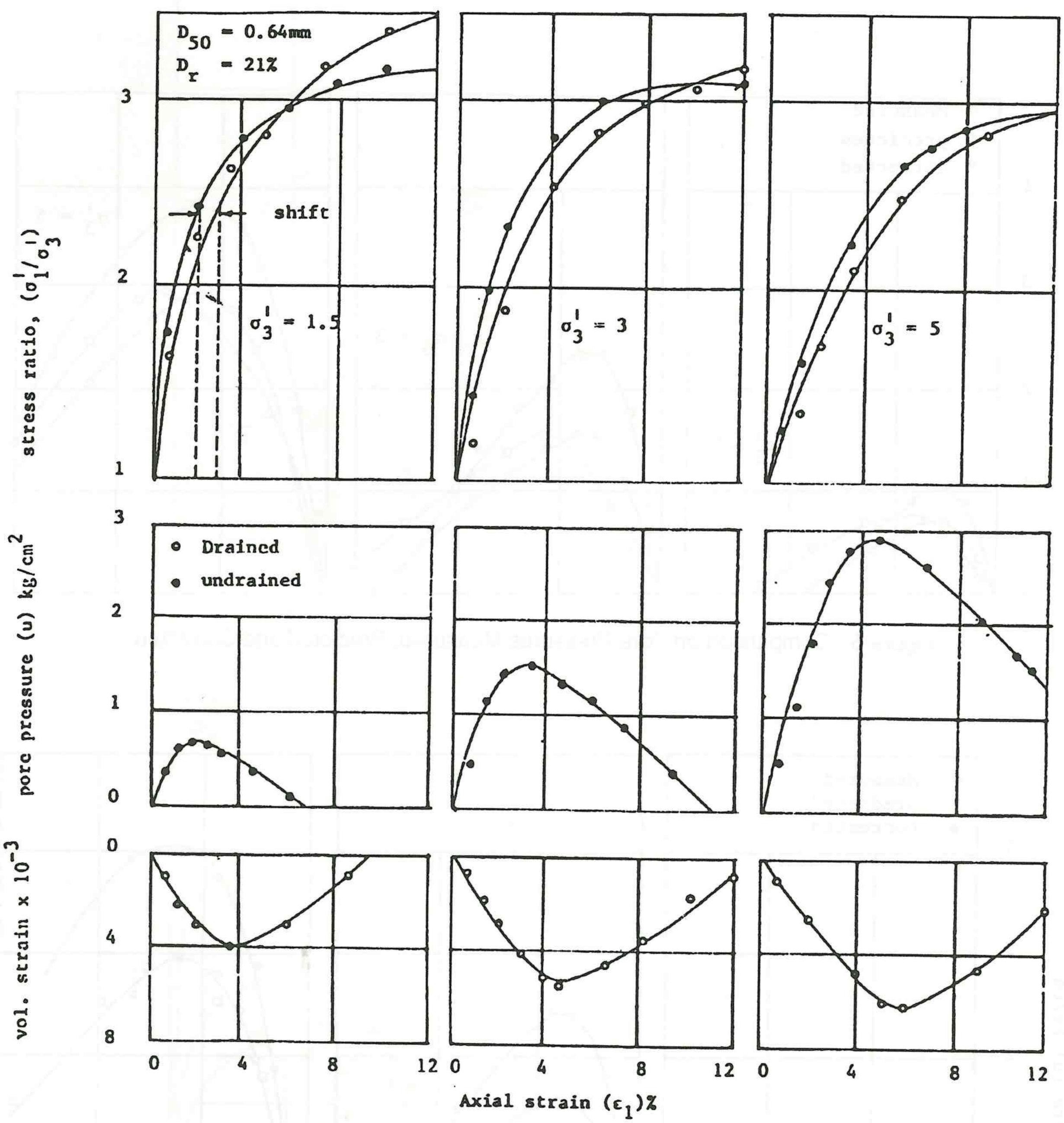


Figure 5 Results of Isotropically Consolidated Drained and Undrained Tests

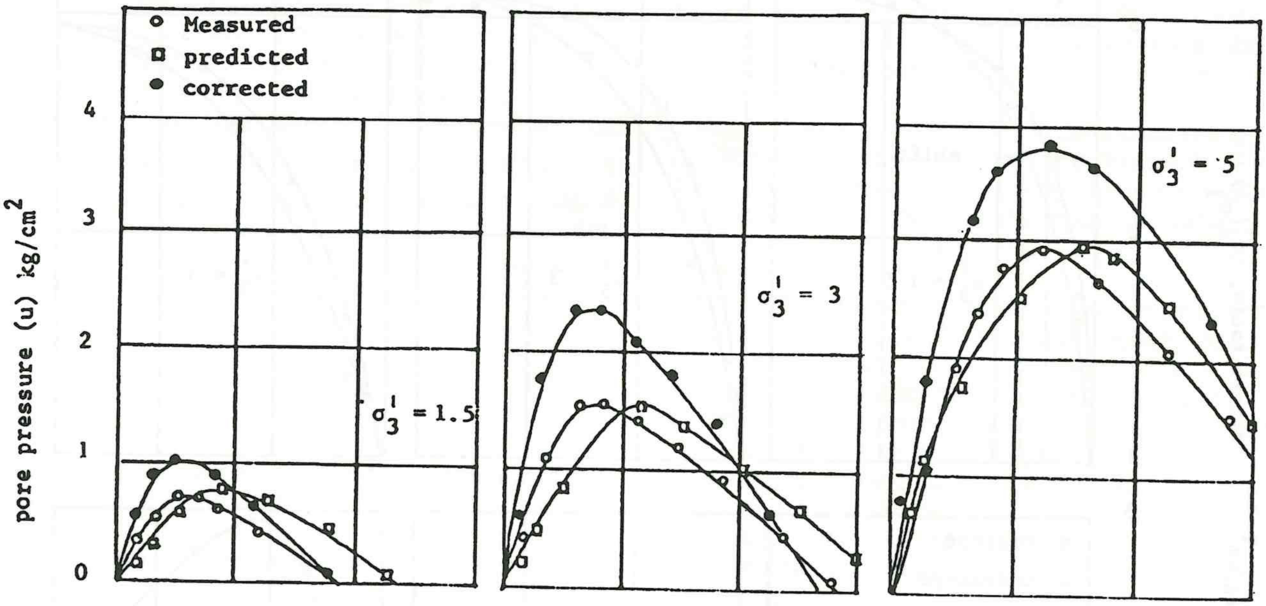


Figure 6 Comparison on Pore Pressures Measured, Predicted and Corrected

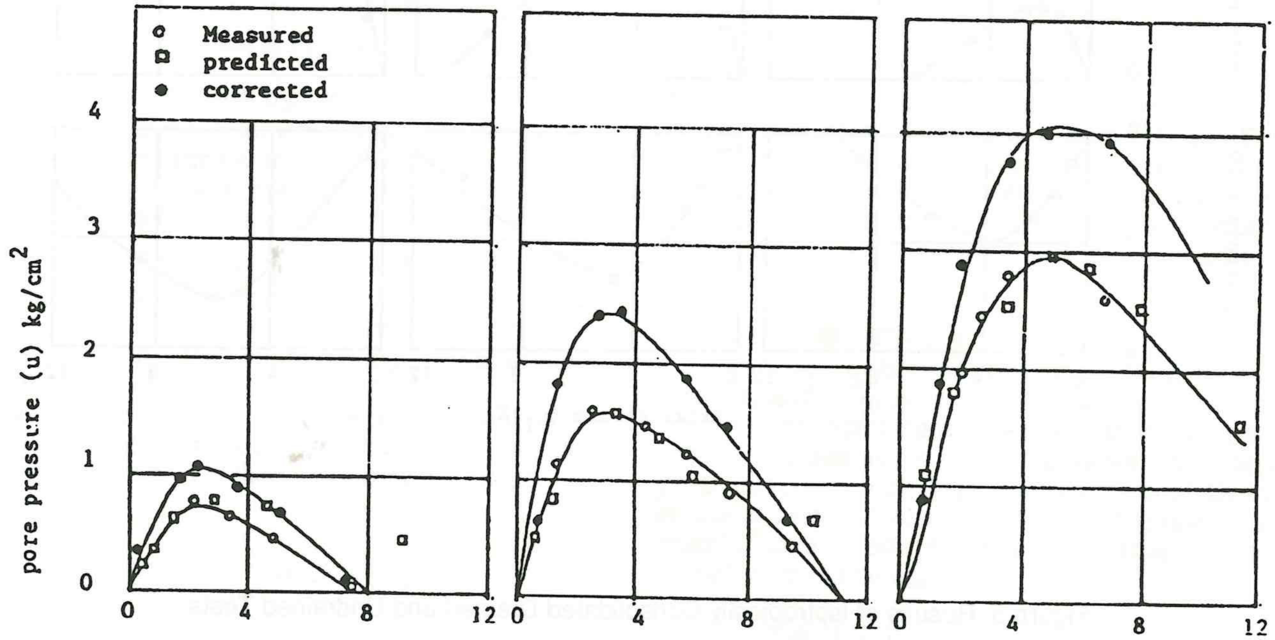


Figure 7 Pore Pressures Corrected for Lateral Shift in Axial Strain

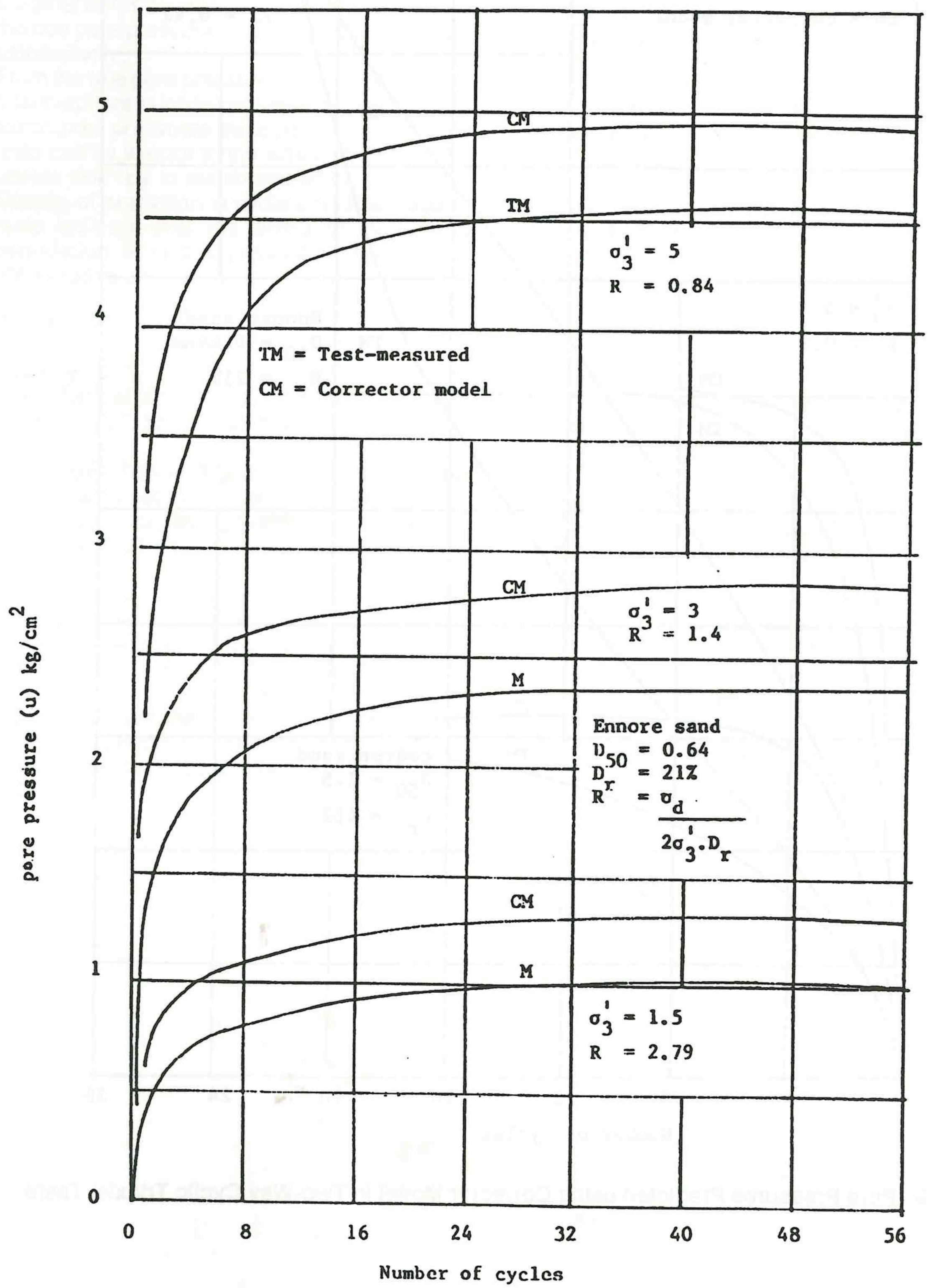


Figure 8 Pore Pressures Predicted using Corrector Model in One-Way Cyclic Triaxial Tests

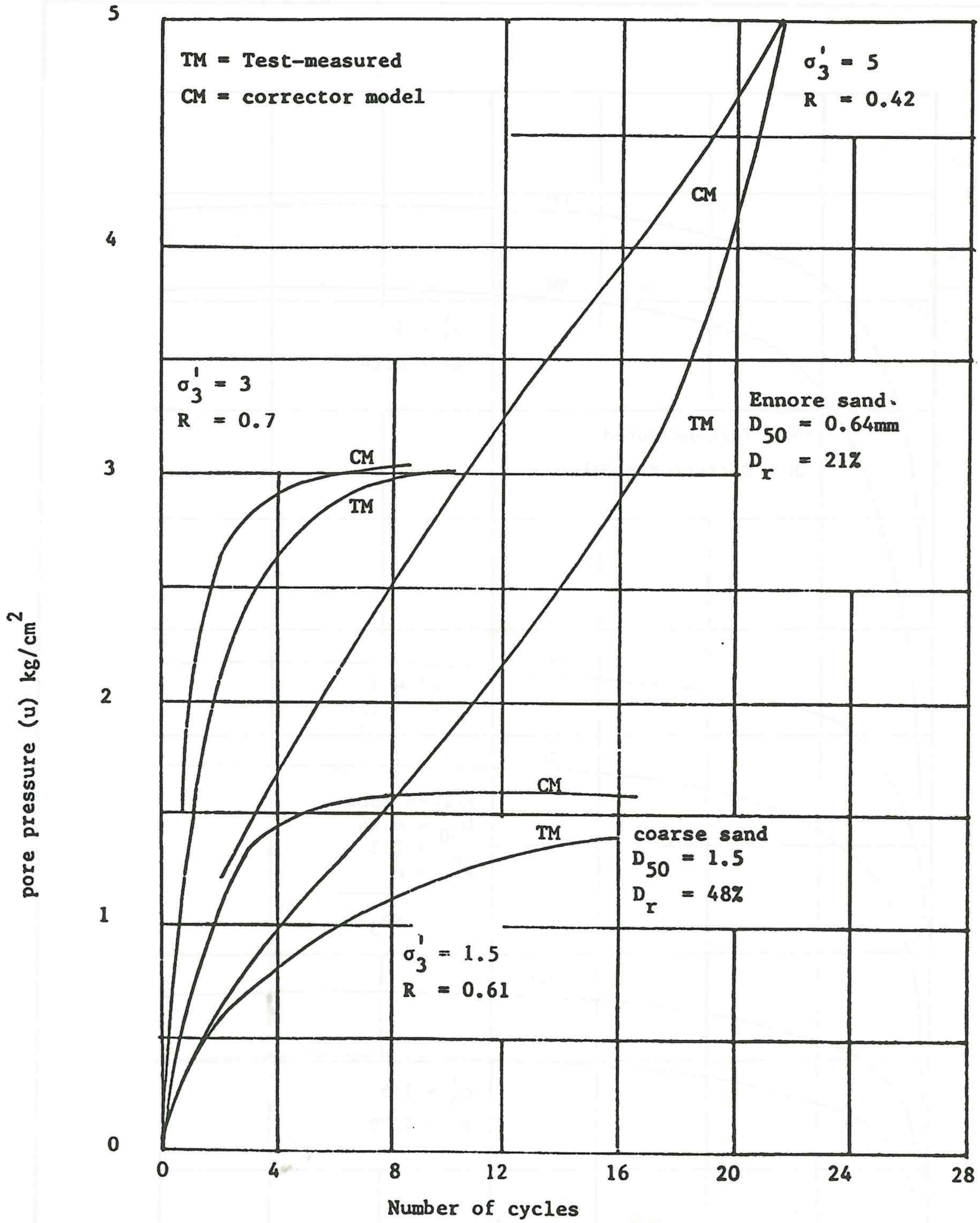


Figure 9 Pore Pressures Predicted using Corrector Model in Two-Way Cyclic Triaxial Tests

models proposed. The results have shown that:

(1) The two analytical models, equations (16) and (17), one representing a corrective procedure and the other a predictive method are found to estimate the true pore pressures in undrained triaxial tests satisfactorily.

(2) From the true pore pressure values predicted it is observed that in loose uniformly graded medium sands pore pressures measured in conventional tests can be in error to the extent of 65% on the unsafe side due to membrane effect.

(3) Results of prediction in cyclic undrained triaxial tests indicate that the effect of membrane penetration is less significant in cyclic tests compared to static tests.

REFERENCES

1. El-Sohby, M.A. (1969). Elastic Behaviour of Sand. Journal of Soil Mech. and Found. Division, ASCE, Vol. 95, SM6, 1393-1409.

2. Frydman, S., Zeitlen, J.G. & Alpan, I. (1973). The Membrane Effect in Triaxial Testing of Granular Soils. Journal of Testing and Evaluation, ASTM, Vol. 1, No. 1, 37-41.

3. Kiekbusch, M. & Schuppener, B. (1977). Membrane Penetration and Its Effect on Pore Pressures. Journal of the Geotechnical Engg. Division, ASCE, Vol. 103, GT 11, 1267-1279.

4. Lade, P.V. & Hernandez, S.B. (1977). Membrane Penetration Effects in Undrained Tests. Journal of the Geotechnical Engg. Division, ASCE, Vol. 103, GT2, 109-125.

5. Newland, P.L. & Allely, B.H. (1959). Volume Changes During Undrained Triaxial Tests on Saturated Dilatant Granular Materials. Geotechnique, Vol. 9, 174-182.

6. Raju, V.S. & Sadasivan, S.K. (1974). Membrane Penetration in Triaxial Tests on Sands. Journal of Geotechnical Engg. Division, ASCE, Vol. 100, GT 4, 482-489.

7. Raju, V.S. & Ramana, K.V. (1980). Undrained Triaxial Tests to Assess Liquefaction Potential of Saturated Sands-Effect of Membrane Penetration. Proc. Int. Symp. on Soils Under Cyclic and Transient Loading, Swansea, Publishers, A.A. Balkema, Rotterdam, Vol. 2, 483-494.

8. Ramana, K.V. & Raju, V.S. (1981). Constant-Volume Triaxial Tests to Study the Effects of Membrane Penetration. Geotechnical Testing Journal, ASTM, Vol. 4, No. 3, 117-122.

9. Ramana, K.V. & Raju, V.S. (1982). Membrane Penetration in Triaxial Tests. Journal of Geotechnical Engg. Division, ASCE, Vol. 108, GT 2, 305-310.

10. Seed, R.B., Anwar, H.A. and Nicholson, P.G. (1989). Elimination of Membrane Compliance Effects in Undrained Testing of Gravelly Soils. Proceedings of the 12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, Brazil, Vol. 1, pp. 111-114.