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A DEGRADATION MODEL FOR THE TRIAXIAL CYCLIC LOADING BEHAVIOR OF SHIRAZ SILTY CLAY*

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Abstract- In this paper the principles of the Ramberg-Osgood expression and the Masing criterion for cyclic loading are described. Modifications to the Ramberg-Osgood expression are reviewed and suggested methods for parameter determination in this equation are presented. The application of the expression for first cycle behavior is illustrated. A common degradation criterion is described and, by representing other researchers' test results, it is shown that the criterion might not be generally applicable. A new degradation model is proposed which is found to be more representative than the other model. A model for predicting cyclic pore pressures for normally consolidated soil is also described. The parameters required for this model have been described in detail and its applicability in predicting cyclic behavior of Shiraz silty clay is presented.

Keywords – Cyclic loading, degradation, cyclic pore pressure

1. INTRODUCTION

To predict the behavior of soils under monotonic and cyclic loading conditions, sophisticated soil models have been proposed. These models, however, require definition of many parameters, which are mostly difficult to obtain. Critical state soil mechanics, for instance, has been used for modelling the monotonic loading behavior of soils, and attempts have also been made to model cyclic and random loading/unloading of soils using this concept [2-4]. These models often fail to predict the various features of soil behavior under cyclic loading. Simple empirical models appear to better simulate these behaviors [5].

A number of empirical models introduced for modeling soil behavior under cyclic loading are the models proposed by Idriss *et al.* [1]; Thiers and Seed [6]; Idriss *et al.* [7]; and Vucetic and Dobry [8]. Among these, the Idriss *et al.* model [1] for normally consolidated soils behavior under strained-controlled cyclic loading conditions seems to have the advantage of simplicity and adequate accuracy. The model basically consists of:

I. An initial stress-strain model using Ramberg-Osgood equation.
II. A modified Masing criterion to form arbitrary hysteretic loops.
III. A criterion for degradation of the initial stress-strain curve under cyclic loading.

In the model, the stiffness reduction is presented as a function of the number of loading cycles and of a degradation parameter, depending on shear strain amplitude. The model, however, provides no explanation of the physical nature of the degradation and it is consistent with the stress-strain behavior only at one point in each loading cycle. Therefore it is unable to account for the degradation

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within the cycle [5]. The excess pore water pressure developed during cyclic loading has not been considered in the model and should be accounted for separately.

2. RAMBERG-OSGOOD STRESS-STRAIN EQUATION

The Ramberg-Osgood equation was first developed to express the stress-strain behavior of material other than concrete and soils [9] and it was used by structural engineers [10] prior to being adopted for soil [11].

The Ramberg-Osgood equation considers the nonlinearity, which usually is observed in stress-strain behavior of soils under monotonic and cyclic loading. In order to model soil behavior under cyclic loading, a specification of the stress-strain relationship during the first cycle is required. Stress-strain curves under cyclic loading may be represented by the trace of the "tips" of all stress-strain loops with different values of cyclic strain, thus defining the *backbone curve* for the soil. The tips are the reversing points in every unloading and reloading loop, such as A, C, A' and C' in Fig.1. It is shown that the backbone curve equation can be rewritten [1] for the parameters involved in the triaxial test as Eq. (1):

$$\frac{\epsilon}{\epsilon_y} = \frac{q}{q_y}\left[1+\alpha\left|\frac{q}{q_y}\right|^{R-1}\right] \qquad (1)$$

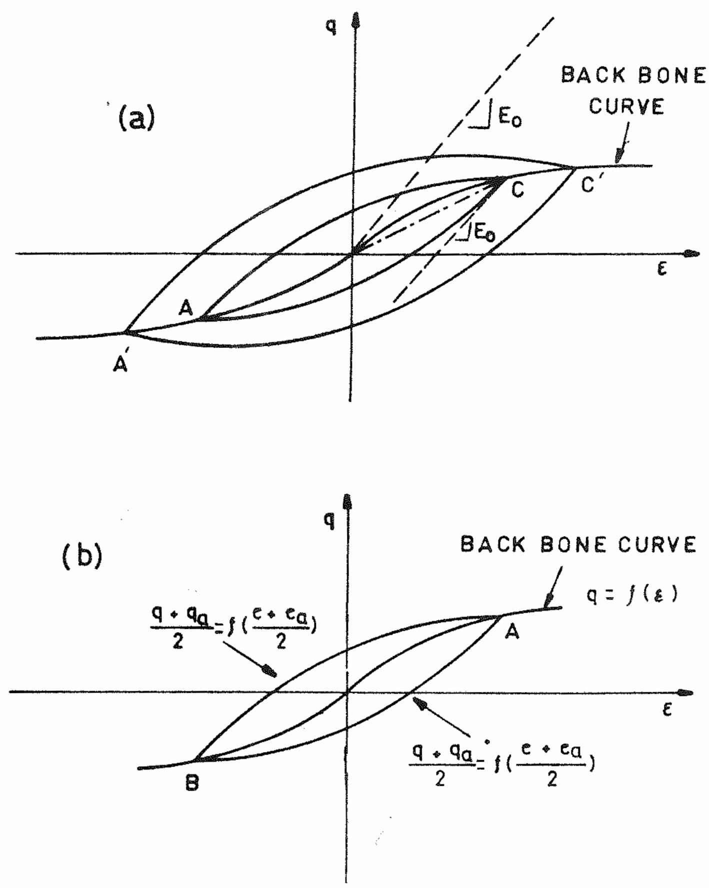

Fig. 1. (a) Backbone curve and (b) hysteresis loops that accord with Masing rule

Where ε_y is appropriate (reference) axial strain, q_y is appropriate (reference) deviatoric stress, ε: is axial strain, q is deviatoric stress and α and R are constants.

The reference (appropriate) shear strain and deviatoric stress may be determined in the following ways. To determine the reference stress and strain, Richart [12] proposed the following quantities for reference stress and strain:

$$q_y = C_1 q_f \tag{2}$$

and

$$\varepsilon_y = \frac{C_1 q_f}{E_0} \tag{3}$$

In which E_0 is the initial or maximum modulus of elasticity and C_1 is a coefficient. Substitution of Eqs. (2) and (3) in Eq. (1) produces Eq. (4).

$$\varepsilon = \frac{q}{E_0}\left[1+\alpha\left|\frac{q}{C_1 q_f}\right|^{R-1}\right] \tag{4}$$

Setting Richart's coefficient C_1 to unity, Hara [13] proposed an alternative form of the Ramberg-Osgood expression, Eq. (5):

$$\varepsilon = \frac{q}{E_0}\left[1+\alpha\left|\frac{q}{q_f}\right|^{R-1}\right] \tag{5}$$

To express the stress-strain loops, however, the Masing rule is commonly used. This criterion stipulates that the unloading and reloading branches of the loop have the same shape as the backbone curve, with both stress and strain scales expanded by a factor of two, and the origin translated. This agrees with experimental evidence for a variety of soils [1]. Let the backbone curve be presented by Eq. (6):

$$q = f(\varepsilon) \tag{6}$$

Referring to Fig.1, if loading reversal occurs at point A where $q=q_a$ and $\varepsilon = \varepsilon_a$, then Eq. (7) describes the subsequent unloading branch of a loading cycle. It is also assumed that after reaching the tips both stress and strain are described by the backbone curve.

$$\frac{q-q_a}{2} = f\left(\frac{\varepsilon-\varepsilon_a}{2}\right) \tag{7}$$

3. DETERMINATION OF THE RAMBERG-OSGOOD EQUATION PARAMETERS

To establish the Ramberg-Osgood equation for predicting the backbone curve in the simplest form, five different parameters should be determined: these are E_0, q_f, α, C_1 and R.

The value of initial modulus could be determined by conducting laboratory or *in situ* tests. In the field, the wave propagation test can give a measure of the modulus. The laboratory tests include the resonant column test, wave propagation test (e.g ultrasonic test), triaxial testing with special deformation measurement devices, etc. Empirical formulae are also available which can be used to estimate the value of initial modulus. These formulae were originally written for shear modulus determination and can be converted for modulus of elasticity E using the following relationship, obtained from the theory of elasticity:

$$E_0 = 2G_0(1+\mu) \tag{8}$$

where G_0 is initial shear modulus and μ is soil's Poisson's ratio. Hardin and Black [14], for instance, proposed Eq. (9) for estimating the value for the initial shear modulus of low plasticity clays and stiff clays having voids ratios between 0.6 and 1.5:

$$G_0 = 3270\left[\frac{(2.97-e)^2}{(1+e)}\right](\sigma_0)^{0.5} \tag{9}$$

in which σ_0 is effective confining pressure (kPa) and e is soil voids ratio. For clays with a high plasticity index, and therefore with high compressibility, Marcuson and Wahls [15] suggested Eq. (10) to estimate the initial modulus of elasticity:

$$G_0 = 445\left[\frac{(4.4-e)^2}{(1+e)}\right](\sigma_0)^{0.5} \tag{10}$$

This equation is more suitable for soft clays with voids ratios between 1.5 to 2.5 [16]. Kokusho *et al.* [17] recommended using Eq. (11) to estimate the initial modulus for soft natural clays.

$$G_0 = 90\left[\frac{(7.32-e)^2}{(1+e)}\right](\sigma_0)^{0.6} \tag{11}$$

The value of shear strength may be measured by conducting triaxial or simple shear tests on the soil. The shear strength can either be determined under static or dynamic conditions. The critical state parameters might be used to estimate shear strength. Different methods have also been suggested for estimating a value for α, the curve shape adjustment factor [13, 16]. Reported values for the parameter α ranges from 5 to 50 [16]. The reported values for R are in the range 1.65 to 3.5 for clays [1, 16]. The value for coefficient C_1 which expresses the relation between reference stress and strain is obtained after determination of the other parameters, so as to produce the best mathematical fit for the back-bone curve vis-a-vis that determined experimentally.

4. MODELS FOR STRESS-STRAIN LOOPS AND SOIL DEGRADATION

Having established the parameters of the Ramberg-Osgood equation, it is possible to express the backbone curve mathematically. Assuming, then, the soil follows the Masing rule, this is applied to the equation for the backbone curve, thus providing the functional expression for the hysteresis loop during the unload-reload cycles, Eq. (12):

$$\varepsilon \pm \varepsilon_c = \frac{q \pm q_c}{E_0}\left[1+\alpha\left|\frac{q \pm q_c}{2C_1 q_f}\right|^{R-1}\right] \tag{12}$$

These derived loops then may be compared with those obtained experimentally as a means of assessing the validity of the Masing criterion in this context. The backbone curve and the hysteresis loops obtained from the above equations are then related to the first cycle of any specified strain of a cyclic loading test.

The behavior of soil under cyclic loading beyond the first few cycles will change due to the degradation of both stiffness and strength. To establish a comprehensive mathematical model it is therefore necessary to be able to express soil degradation behavior. To achieve this, use can be made of the experimental results after the first cycle and then relate these to either an empirical or theoretical rule for behavior thereafter [1, 18].

A mechanism has been expressed by Idriss *et al.*[1] for this purpose and confirmed by other researchers (e.g., [8, 18]). They presented a degradation index, δ, and a degradation parameter, t and defined the degradation index as follows:

$$\delta = \frac{(E_S)_N}{(E_S)_1} \tag{13}$$

In the above equation $(E_S)_N$ is the secant modulus of elasticity at a specified cycle number (N). This ratio (i.e the degradation index) depends on the number of cycles and the cyclic strain in a strain-controlled test. Idriss *et al.* [1] found a linear relation between δ and N when plotted on a logarithmic scale, thus they proposed the following empirical relationship between the degradation index and the number of cycles:

$$\delta = N^{-t} \tag{14}$$

In this equation, t is defined as a degradation parameter, and is dependent on the cyclic strain amplitude. Other researchers have tried to relate t to specified properties of the soil. Moriwaki and Doyle [18], for example, proposed the following empirical relation between t and cyclic strain.

$$t = \frac{\varepsilon_c}{a + b\varepsilon_c^x} \tag{15}$$

In which a, b and x are constants determined experimentally. Vucetic [19] confirmed the linear relation between logarithms of δ and N, not only for normally consolidated, but also for over-consolidated soils. He found that the degradation parameter depended on the over-consolidation ratio, as well as cyclic strain, and in such a way that t consistently increased with increasing cyclic strain and decreased with an increase in over-consolidation ratio. He showed that the degradation parameter can be plotted versus the above factors or can be related to them using mathematical expressions.

According to Idriss *et al.* [1] this concept provides a mechanism for establishing the backbone curve for the second, third and Nth cycle (i.e. *degraded curve*) as well as for the first cycle (i.e. the non-degraded curve). Thus it is possible to relate the tip coordinates, q_N, of a hysteresis loop in cycle N to that measured in the first cycle at a specified strain, using Eq. (16).

$$\delta = \frac{q_N}{q_1} \tag{16}$$

Thus the entire backbone curve (i.e. for all strain levels) in cycle N can be related to the first cycle backbone curve. Accordingly, using the Ramberg-Osgood formulation the backbone curve in cycle N can be formulated as follows:

$$\varepsilon = \frac{q}{\delta E_0}\left[1 + \alpha\left|\frac{q}{\delta C_1 q_f}\right|^{R-1}\right] \tag{17}$$

The unloading and reloading branches of the hysteresis loops can also be constructed using equation (18).

$$\varepsilon \pm \varepsilon_c = \frac{q \pm q_c}{\delta E_0}\left[1 + \alpha\left|\frac{q \pm q_c}{2\delta C_1 q_f}\right|^{R-1}\right] \tag{18}$$

5. THE EQUIPMENT AND TEST PROGRAM

A cyclic triaxial apparatus was developed at Shiraz University to investigate the behavior of Shiraz silty clay under monotonic and cyclic loading. The apparatus used has been described elsewhere [20].

A test program was established and the results of the monotonic and cyclic loading tests were analyzed to define various parameters for the soil. Undistributed specimens of soil for testing were obtained using a special thin tube soil sampler, designed and built at Shiraz University [21]. The characteristics of soil are presented in Table 1 and the average grain size curve is shown in Fig.2.

Table 1. The characteristics of the soil tested

Liquid Limit (%)	33-38
Plastic Limit (%)	10-15
Specific Gravity (Gs)	2.78
Passing # 200 sieve	95

To obtain the soil parameters for monotonic loading, three conventional consolidated undrained compression tests were conducted using isotropical consolidation pressures of 50, 150 and 200 kPa. All tests were strain-controlled and the rate of deforming was 0.18 mm/min. An over-consolidated sample with an over-consolidation ratio of 8 was also tested under a cell pressure of 25 kPa. The results are shown in Fig.3.

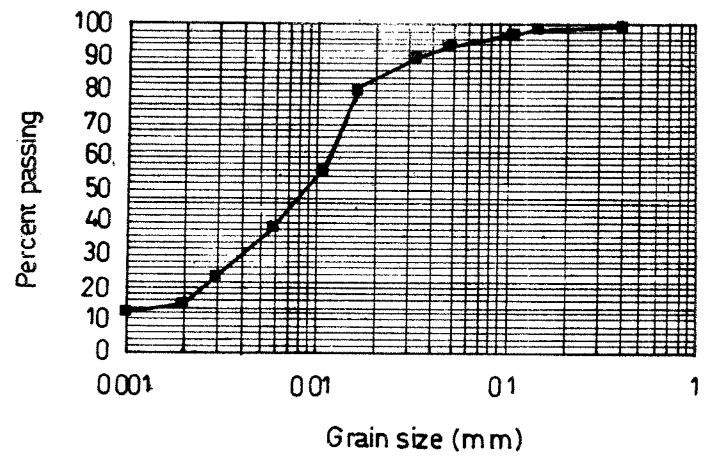

Fig. 2. Average grain size of the tested soil

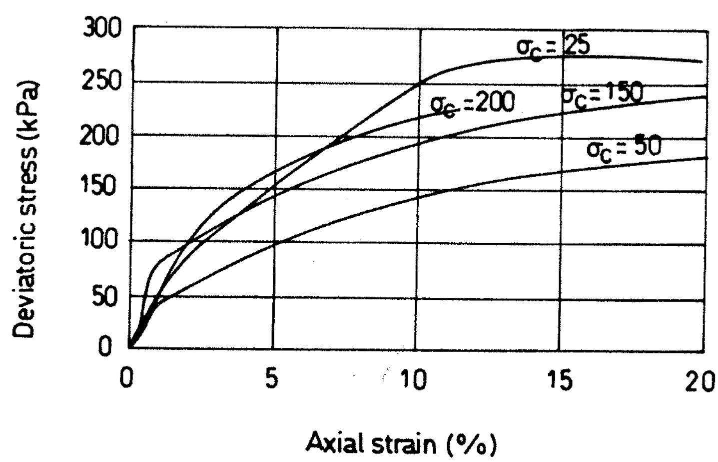

Fig. 3. Resulted stress strain curves from monotonic loading tests

Triaxial extension tests were also performed under consolidation pressure of 50 to 255 kPa. The results are presented in Fig.4.

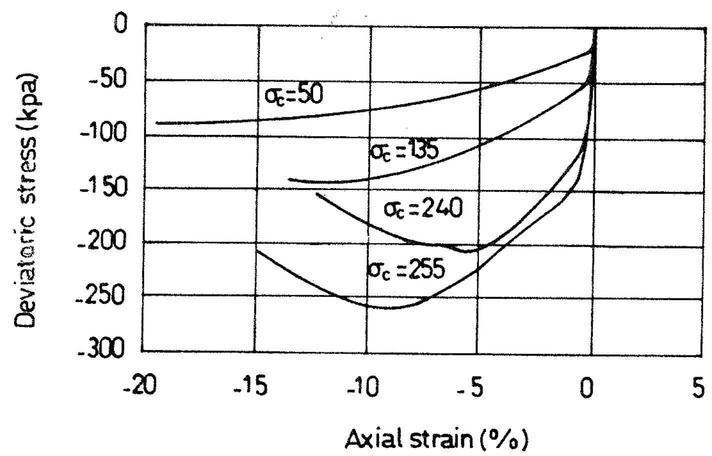

Fig.4. Resulted stress strain curves from monotonic extension tests

A series of strain-controlled cyclic loading tests with cyclic strain amplitudes of 0.1, .5, 2 and 3 percent were also performed. Typical results are shown in Figs. 5 and 6. The full description of test procedure is given by Aghaebrahimi [21].

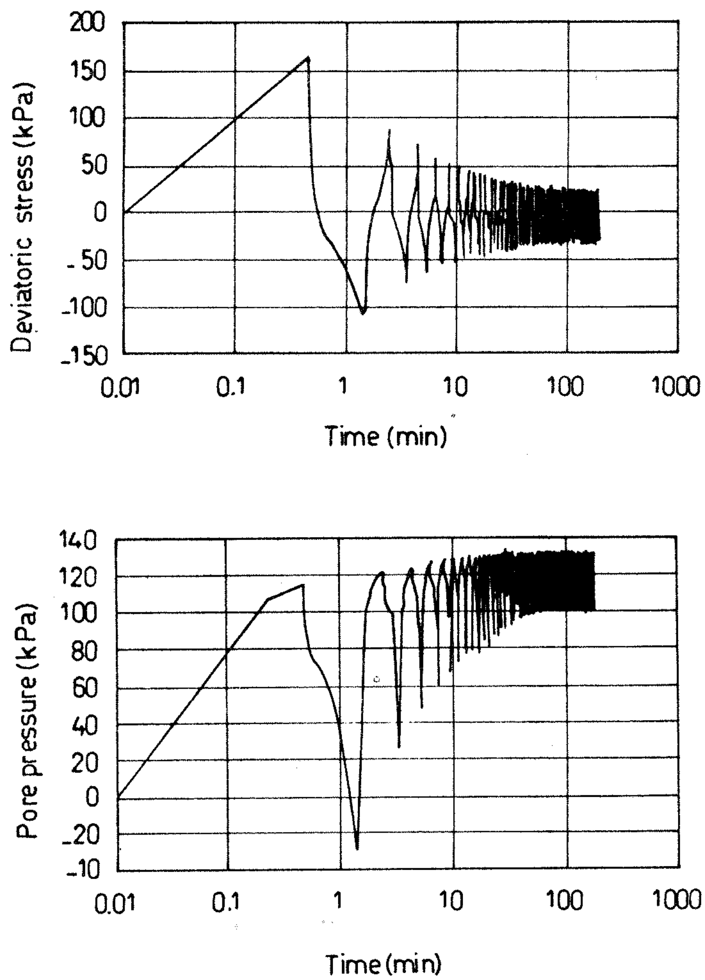

Fig. 5 a & b, Typical cyclic loading test results (strain amplitude=3%)

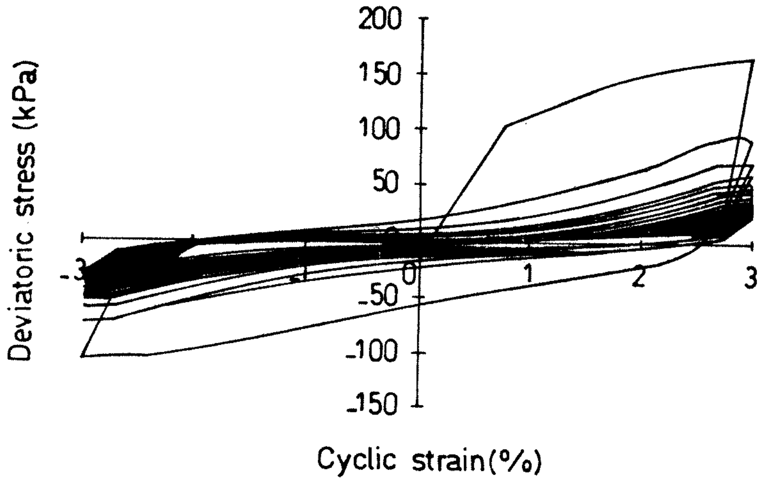

Fig. 5 c, Typical cyclic loading test results (strain amplitude=3%)

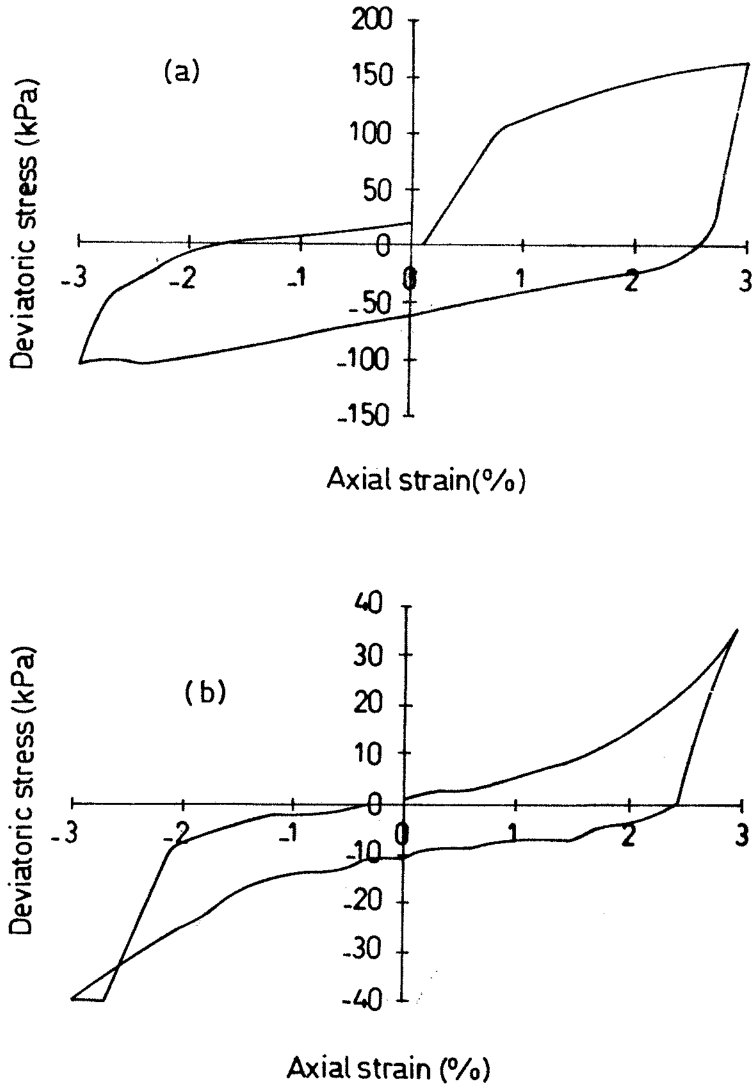

Fig. 6. Hysteresis loops: (a) N=1, (b) N=10

6. DEVELOPMENT OF THE RAMBERG-OSGOOD EXPRESSION FOR SHIRAZ SILTY CLAY

The results of triaxial cyclic loading tests on undisturbed specimens of Shiraz silty clay were used to establish the expression for the backbone curve and to develop the basis of the Ramberg-Osgood equation. As discussed previously, in the Ramberg-Osgood equation five parameters need to be determined. The values of these parameters are determined using the procedures mentioned, and the Ramberg-Osgood expression for Shiraz clay is proposed as follows:

$$\varepsilon = \frac{q}{1000}\left[1 + 46\left|\frac{q}{200}\right|^{2.27}\right] \tag{19}$$

The adequacy of this expression is verified by comparing the predictions of behavior obtained from Eq. (19) with the experimental results, as shown in Fig. 7.

Having established a mathematical expression for the backbone curve, it is now possible to predict the behavior of the soil under cyclic loading conditions. The backbone curve equation predicts the stresses at the tips of the stress-strain loops at different cyclic strain amplitudes. Assuming the adequacy of the Masing criterion, it should then be possible to model the stress-strain loops for each cyclic strain amplitude during the first cycle. This is shown in Fig.8, in which the predicted response of soil is again compared with the experimental results at different cyclic strain amplitudes. As it can be seen for both backbone curve and stress strain hysteresis loops, the agreements are reasonably good.

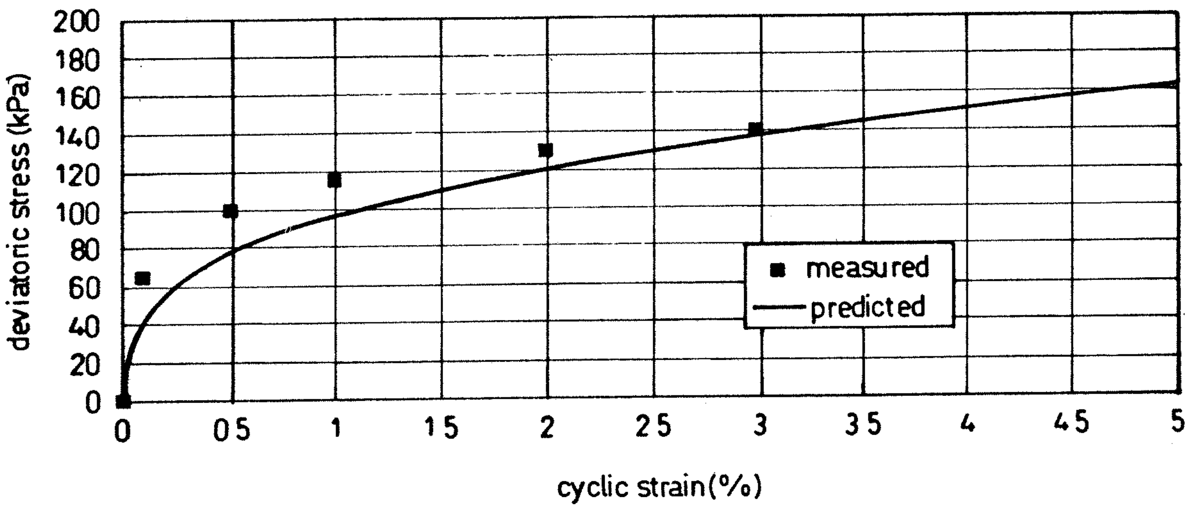

Fig. 7. Comparing the predicted backbone curve with experimental data

7. DEVELOPMENT OF DEGRADATION MODEL FOR SHIRAZ SILTY CLAY

To model soil behavior under cyclic loading beyond the first cycle, a functional relationship between the soil properties and the number of cycles is needed. Using the concept of degradation index relation as a function of N, the results of tests on the specimens are shown in Fig.9. As it can be seen, the relationship between the logarithm of the degradation index (or secant modulus ratio) and the logarithm of the number of cycles is not linear for the soil, despite the expected linear relationship reported by Idriss *et al.* [1]. Other researchers, e.g. Taylor and Hughes [22], Taylor and Bacchus [23], and Procter and Khaffaf [24]; have also reported a nonlinear relationship between these parameters, thus suggesting that the proposed relation might not be generally applicable. Taylor and Hughes [22] reported that the rate of secant modulus fall-off decreased after a comparatively small number of cycles in strain-controlled cyclic loading tests. They also reported that for some cases the modulus became nearly constant, having fallen to approximately 50 % of its initial value after only 40 cycles. Taylor and Bacchus [23] carried out a series of strain-controlled cyclic tests on Halloysite clay. Cyclic strain amplitudes equal to 0.3, 0.8 and 1.67 were used. Whilst it was observed that the largest cyclic strain produced the largest stiffness degradation rate with N, the rate of degradation reduced with the number of cycles, tending towards an equilibrium value.

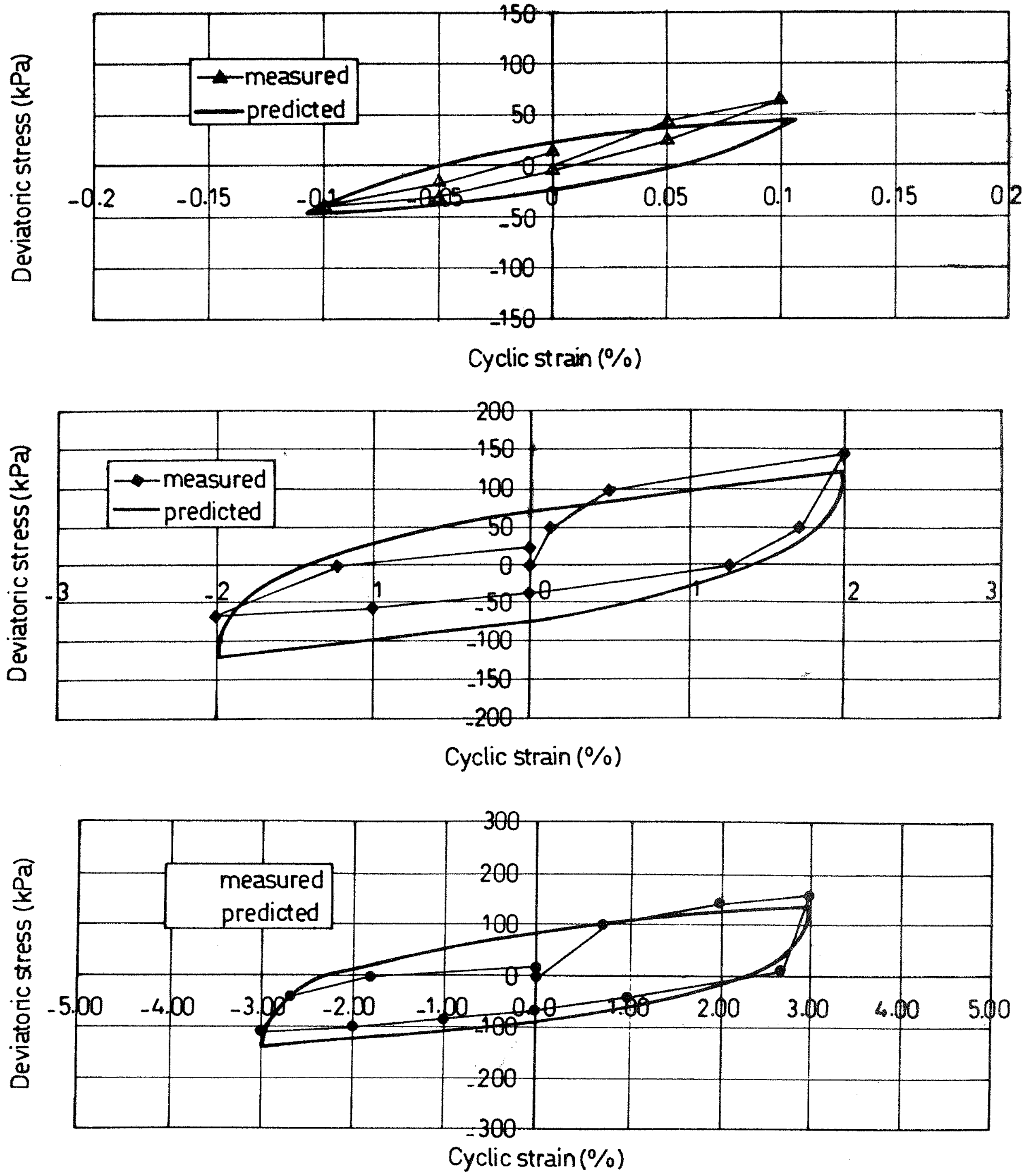

Fig.8. Cyclic stress-strain loops for different strain amplitude

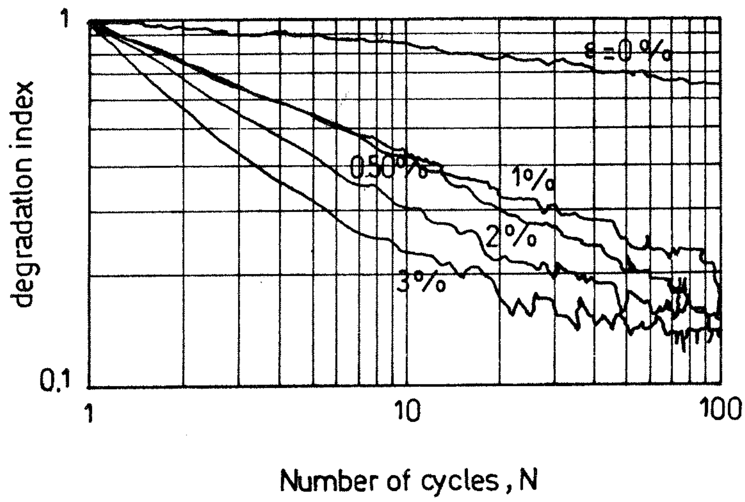

Fig. 9. Variation of degradation index with N, experimental results

Procter and Khaffaf [24], in a series of strain-controlled tests on saturated remolded Derwent clay, also observed a decreasing rate of softening with an increasing number of cycles. Furthermore, they observed that, irrespective of strain amplitude, after a certain number of cycles the soil reached a limiting strength. This was especially obvious for frequencies less than 0.2 Hz. A similar phenomenon (i.e. the nonlinear relation between degradation index and *N*) was reported by Sherif *et al.* [25]. These observations therefore confirm that the criterion proposed by Idriss *et al.* [1] might not generally be acceptable for all clayey soils. Thus a modification has been made to the proposed criterion to produce a new degradation criterion which includes both types of behavior. The results of cyclic loading tests on normally consolidated specimens with different cyclic strain amplitudes indicated the existence of a common limit value for tip deviatoric stresses. This is in agreement with the reported results of tests on Derwent clay [24-25].

If this limiting value is referred to as q_l, a relationship between the degraded deviatoric stress q_N and cycle number N may be expressed by Eq. (20):

$$q_N = q_1 N^{-t_m} + q_l \tag{20}$$

where t_m is a modified version of the t parameter referred to by Idriss *et al.* [1] as the degradation parameter. This equation, of course, is used for values of N greater than unity since the value of q at N=1 can be determined from the backbone curve equation. The first part of the right-hand side of this equation tends toward zero with increasing N due to the negative sign of t_m, and q_l becomes the dominating value. Values of t_m might be determined experimentally for different cyclic strain amplitudes and interpolated for other strain levels, or t_m may be evaluated using mathematical expressions.

A modified degradation index δ_m, can similarly be redefined by Eq. (21).

$$\delta_m = \frac{q_N}{q_1} = N^{-t_m} + \frac{q_l}{q_1} \tag{21}$$

The empirical relation proposed by Idriss *et al.* [1] is, of course, a particular case of the above relation with q_l equal to zero. It should now be possible to make more realistic predictions of the cyclic behavior of the soil.

8. VERIFICATION OF THE MODIFIED CRITERION

In order to examine the modified degradation criterion, the data obtained through cyclic triaxial tests on Shiraz clay are employed and compared with the values produced by Eq. (20). This has necessitated determination of values for q_l/q_1 and t_m for the soil. Subsequently a value of 0.15 (at large number of loading cycles) was chosen for q_l/q_1. The values of t_m at different strain amplitudes were determined experimentally and are shown in Fig.10. It can be seen that the values of t_m increase with an increase in cyclic shear strain. This is in agreement with the findings of Vucetic *et al.* [19]. Also included in this figure is the graph of predicted t_m versus cyclic strain using the following relationship:

$$t_m = \frac{1}{(1+\frac{1}{\varepsilon_c^{0.7}})} \tag{22}$$

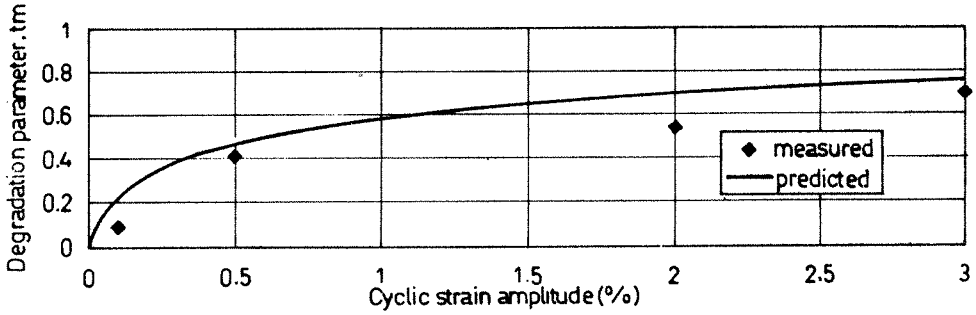

Fig. 10. The obtained and predicted values of t_m

The agreement between the experimental data and predicted curve is reasonably good.

The concept may therefore be used to predict the variation of secant modulus of elasticity of soil with number of cyclic loading at different strain amplitude using Eq. (23).

$$E_N = E_1\left(N^{-\frac{1}{(1+\frac{1}{\varepsilon_c^{0.7}})}} + .15\right) \tag{23}$$

This is performed and the typical results for cyclic strain of 3% are shown in Fig.11. The experimentally obtained data are also included in this figure and agreement is acceptable, which indicates the ability of the modified criterion to predict the degradation behavior of these soils, whilst the Idriss *et al.* [1] criterion was found unsatisfactory in this context.

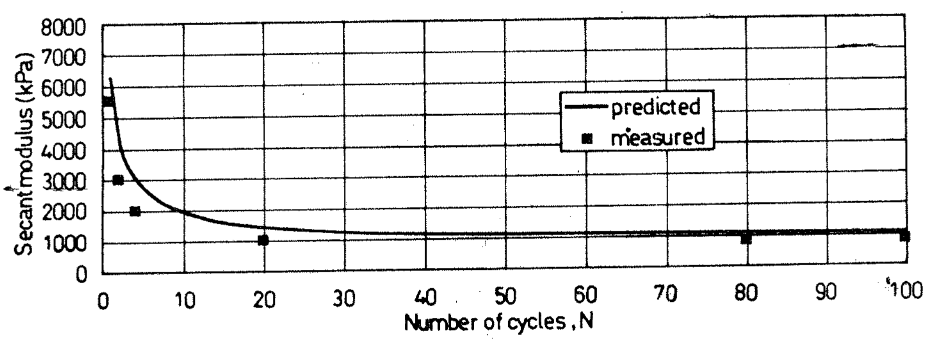

Fig.11. Changes of secant modulus with number of cycles

9. CYCLIC PORE PRESSURE MODEL

The above procedure for modeling soil behavior under cyclic loading is based on deviatoric stresses only, and pore pressure and effective stresses have not been considered. To extend the proposed model to other aspects of soil behavior, one should be able to model the pore pressure behavior of soil specimens under cyclic loading.

The variation of pore pressures during cyclic loading of the soil is predicted using a relationship proposed by Hataf, [26]. He reported a linear relationship between normalized pore pressure at Nth cycle number, U_N^* and log (N). This relation may be expressed as:

$$U_N^* = U_1^* + \beta \log(N) \tag{24}$$

where U_1^* is the value for the dynamic pore pressure at N=1 normalized with respect to the initial consolidation pressure, and β is the slope of the lines (independent of the cyclic strain amplitude). The values of β for Shiraz silty clay, listed in Table 2, were obtained experimentally and are shown in Fig.12 for different cyclic strain amplitudes. As it can be seen, the value of both β and U_1^* for the soil are not independent, and are functions of cyclic strain amplitude which can be determined experimentally.

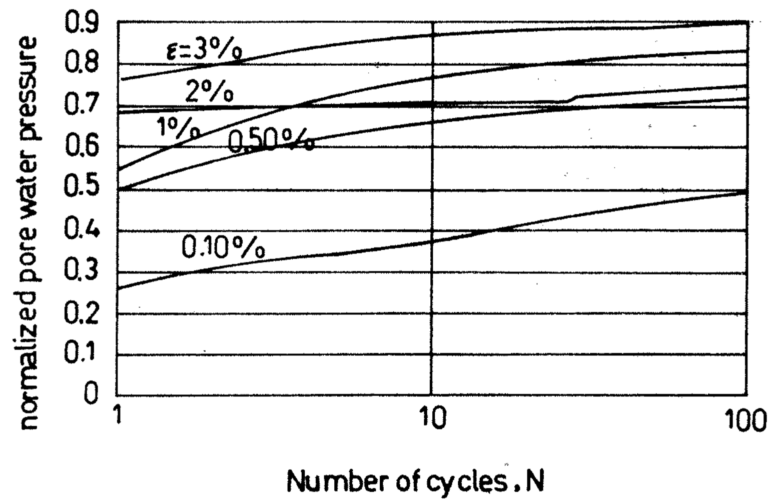

Fig. 12. Changes of normalized pore water pressure with N

Table 2. The values of parameter β for different cyclic strains

Cyclic strain (%)	β
0.1	-0.13
0.5	0.12
1	0.07
2	0.04
3	0.04

The plots of normalized pore pressure change rate (U_N^* / N) with a logarithm of N for the soil presented in Fig.13. showed an approximately linear relationship which may be considered independent of cyclic shear strain amplitude as follows:

$$log(\frac{U_N^*}{N}) = C + w\, log(N) \tag{25}$$

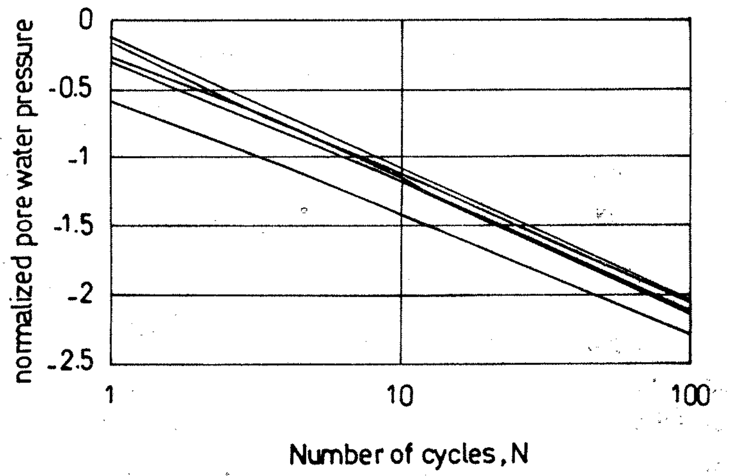

Fig.13. Variation of normalized pore water pressure with respect to N versus log(N)

The average values of w and C for this soil were -0.912 and 0.23, respectively. Using the above relationship eases the mathematical modeling because there is no need to know the pore pressure values for a given cyclic strain amplitude at $N=1$.

Therefore, principal effective stresses of soils may be obtained if the values of deviatoric stresses and pore pressures are known. In the proposed model, using the principle of effective stress, once a functional relation between stress and pore pressure under cyclic loading and N is established, it is possible to predict the mean effective stress at any cycle under undrained conditions from Eq. (26).

$$P'_N = \frac{\sigma'_{aN} + 2\sigma'_{rN}}{3} = \frac{\sigma_{aN} - 2\sigma_{rN}}{3} - U_N + \sigma_{rN}$$

$$P'_N = \frac{q_N}{3} - U_N + \sigma_{rN} \tag{26}$$

$$P'^*_N = \frac{P'_N}{\sigma_{rN}} = \frac{q_N^*}{3} - U_N^* + 1$$

Here, $\sigma'_{aN}, \sigma_{aN}$ are effective and total axial stresses, $\sigma'_{rN}, \sigma_{rN}$ are effective and total radial stresses at any N>1 cycles, P'^*_N , q'^*_N and U^*_N are effective mean stress, deviatoric stress and pore pressure normalized with respect to the initial consolidation pressure. Substituting the equivalent equations for q'^*_N and U^*_N yields to Eq. (27) or (28):

$$P'^*_N = \frac{q_1^* N^{-t_m}}{3} - \beta\, log(N) + \left(\frac{q_I}{3} - U_1^* + 1\right) \tag{27}$$

$$P'^*_N = \frac{q_1^* N^{-t_m}}{3} - w\, log(N) + \left(\frac{q_I^*}{3} - C + 1\right) \tag{28}$$

Therefore, it should now be possible to model different aspects of behavior for the soil under cyclic loading using the foregoing relations.

10. CONCLUSION

The Ramberg-Osgood expression for stress-strain behavior of soils and the Masing criterion for cyclic loading, along with the results of laboratory tests, were used to model the first cycle stress-strain behavior of Shiraz silty clay soil. A new degradation model was proposed to predict the behavior of the soil beyond the first cycle. The proposed model for normally consolidated Shiraz silty clay soil is verified using the results of cyclic loading tests.

Having established the expressions for the backbone curves for the soil, predicted backbone curves were plotted against cyclic shear strain together with the experimental data points. The fitting was reasonably good and therefore it might be concluded that the parameters are adequate to predict the first cycle peak stresses.

The validity of the Masing criterion was verified by comparing the stress-strain loops obtained from the mathematical expression based on this criterion with the data obtained experimentally.

Changes of deviatoric stress and/or secant modulus of elasticity of soil versus N were shown. The values of t_m were obtained using the functional relation mentioned earlier. There is good agreement between the predicted and measured behavior.

A method of modeling cyclic pore pressures for the soil was described and empirical relations expressing pore pressure changes in normally consolidated specimens were obtained. The model may be used for other clayey soils after it has been verified by further testing on such soils.

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