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R. Dyvik

Rensselaer Polytechnic Institute, Troy, NY

T. F. Zimmie

Rensselaer Polytechnic Institute, Troy, NY

P. Schimelfenyg

Rensselaer Polytechnic Institute, Troy, NY

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R. Dyvik, Research Associate
T. F. Zimmie, Associate Professor
P. Schimelfenyg, Research Assistant, Civil Eng. Dept.
Rensselaer Polytechnic Institute, Troy, N.Y.

SYNOPSIS Consolidated, constant volume (CCV), cyclic laboratory shear tests were performed on marine clay soils. The Norwegian Geotechnical Institute (NGI) direct simple shear device, modified for cyclic loading (square wave) capabilities, was used for these tests. Two clays were investigated; a natural undisturbed Gulf of Mexico clay and a reconstituted Pacific Illite. The cyclic shear tests were performed with two way loading (complete stress reversal). Models are presented to predict the pore pressure behavior during a cyclic test, or to predict the strain or pore pressure behavior of tests with varying cyclic shear stress levels. A unique relationship between shear strain and pore pressure for the tests in this investigation is presented.

INTRODUCTION

The cyclic behavior of fine-grained soils subjected to repeated loadings such as those produced by earthquakes, wind, waves and machine vibrations recently has been studied by many researchers (Anderson, et al, 1978, Floess and Zimmie, 1979 and others). Practical and economical limitations require that a few laboratory tests yield a maximum amount of information. Models are required that utilize available test data to predict the cyclic behavior of soils under differing conditions.

Models in this paper are developed from controlled-stress cyclic shear tests on marine clays and can be used to predict the cyclic behavior of the soils at any cyclic stress level. These models can also be used to predict the behavior of the soils subjected to varying cyclic stress levels, as is the case with most natural repeated loading phenomena. Examples are given to demonstrate the application and comparison of the models.

TESTING

All tests were conducted utilizing a Norwegian Geotechnical Institute (NGI) direct simple shear apparatus (model number 4). The device was developed by NGI and is manufactured by Geonor. It was modified so that cyclic stress-controlled tests with square wave loading could be performed (test frequency was 0.25 Hz).

During shear, undrained conditions were simulated by keeping the volume of the specimen constant. A wire reinforced membrane maintained a constant cross sectional area of the specimen. The height of the specimen was kept constant by changing the vertical (normal) stress. This change in vertical stress is assumed to be equal to the change in pore water pressure that would have occurred during an undrained test (Prevost and Høeg, 1976). False deformation of the NGI device as a function of applied stress was taken into consideration to maintain a constant height and to obtain accurate pore pressure measurements.

An undisturbed Gulf of Mexico clay and a reconstituted Pacific Illite were the clays used in the testing program. The Gulf of Mexico clay was cored about 225 miles east of Corpus Christi, Texas at about 1000 feet of ocean depth by the United States Geological Survey. The reconstituted Pacific Illite was dredged about 600 miles north of Hawaii at about 5000 feet of depth by the University of Rhode Island and reconstituted at their Geomechanics Laboratory. Pertinent geotechnical data for the clays is listed below in Table 1.

Table 1 Geotechnical Data

	PACIFIC ILLITE	GULF OF MEXICO CLAY
WATER CONTENT (%)	86-94	70-105
LIQUID LIMIT (AVERAGE,%)	86	105
PLASTIC LIMIT (AVERAGE,%)	30	30
SPECIFIC GRAVITY	2.71	2.71
SENSITIVITY (FALL CONE)	2.3	2.7
ULTIMATE STATIC SHEAR STRENGTH, S_u (Kg/cm^2)	0.145	0.130
MAXIMUM PAST PRESSURE (Kg/cm^2)	0.230	0.240

All specimens were consolidated to a confining stress of 0.518 kg/cm^2 bringing them to a normally consolidated state (overconsolidation ratio of 1.0). After consolidation, complete stress reversal shear tests were conducted at different cyclic shear stress levels.

The wire reinforced membranes (50 cm^2 in area) were calibrated to measure lateral specimen stresses (Dyvik, Zimmie, and Floess, 1980). Lateral stresses were measured for all tests on a routine basis and enabled the coefficient of lateral earth pressure (K_o), Mohr's circles states of stress, stress paths, etc., to be determined (Floess and Zimmie, 1979).

CYCLIC SHEAR RESULTS

Six cyclic shear tests were performed on each of the two clays in this study. The ultimate shear strengths, S_u, were 0.145 kg/cm^2 and 0.130 kg/cm^2 for the Pacific Illite and Gulf of Mexico clay, respectively. The cyclic shear stress level, τ_c, for each test, expressed as a percentage of S_u, is presented in Table 2.

Table 2 Cyclic Stress Levels

PACIFIC ILLITE TESTS						
TEST NUMBER	2	3	5	6	7	8
τ_c/S_u (%)	37	51	58	44	48	51
GULF OF MEXICO CLAY TESTS						
TEST NUMBER	9	10	11	12	13	14
τ_c/S_u (%)	54	54	54	46	46	42

Figure 1 shows the relationship between cyclic shear strain and number of loading cycles for each clay. The cyclic shear strain is one half the peak to peak (left to right) strain. The cyclic shear strain in tests 6 and 14 gradually increased to 0.58 percent and 1.05 percent, respectively, at 3000 cycles. The cyclic shear strain in test 2 stabilized at approximately 0.25 percent at 1500 cycles and remained as such to 3000 cycles (cyclic equilibrium)(Sangrey et al, 1969).

Figure 2 shows the excess hydrostatic pore water pressure buildup with number of cycles for these tests. The excess pore pressure is normalized to the final consolidation stress, $\bar{\sigma}_{vo}$ (0.518 kg/cm^2). Note that the maximum normalized excess pore pressure is approximately 0.8 (determined from the change in applied vertical stress). Excess pore pressures within the specimen were actually measured during consolidation. At the instant of consolidation stress application, the calibrated membrane typically indicated normalized lateral stresses (excess pore pressures) of 0.80 to 0.82. The reason normalized excess pore pressure did not reach unity during consolidation or shear was most likely due to the lack of back pressure (Lambe and Whitman, 1969). The normalized excess pore pressure for tests 2, 6 and 14 reached values of 0.220 kg/cm^2, 0.427 kg/cm^2 and 0.430 kg/cm^2, respectively, at 3000 cycles.

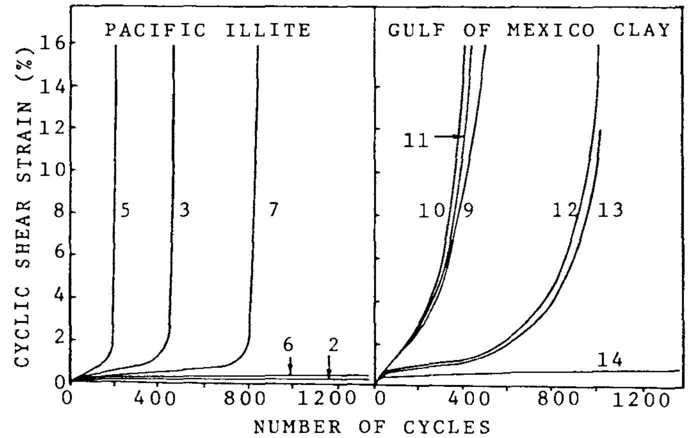

Fig.1 Cyclic Shear Strain versus Number of Cycles

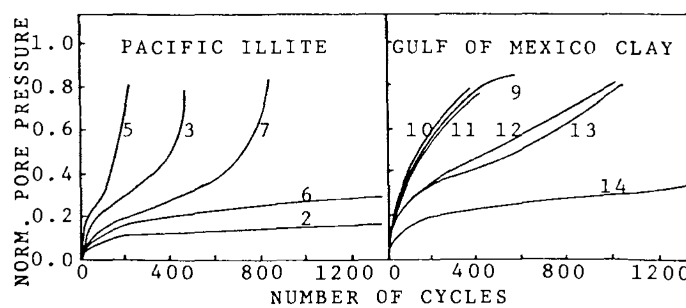

Fig.2 Normalized Pore Pressure versus Number of Cycles

The reproducibility of test results was quite good. Figures 1 and 2 for the Gulf of Mexico clay indicate consistent shear strain and pore pressure behavior for tests performed at similar cyclic stress levels.

MODELLING CYCLIC BEHAVIOR BY STRAIN

Cyclic shear test results are often presented as lines of equal shear strain (strain contours) on axes of cyclic shear stress level and number of cycles. These contours are shown in Figures 3 and 4 for the Pacific Illite and the Gulf of Mexico clay, respectively. The number of cycles to failure increases rapidly with decreasing cyclic shear stress. These contours asymptotically approach a critical level of repeated loading (critical shear stress level), below which the cyclic shear strains are essentially inde-

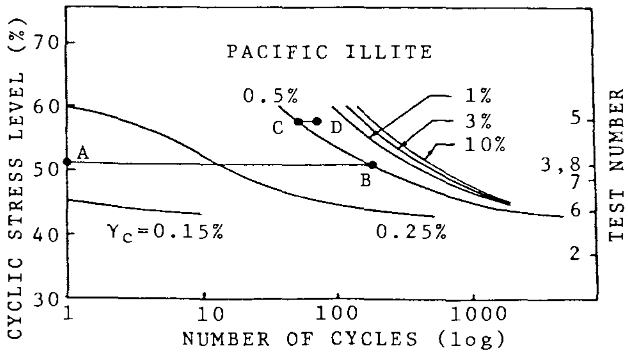

Fig.3 Cyclic Shear Strain versus Cyclic Stress Level and Number of Cycles

pendent of the number of cycles (cyclic equilibrium) (Sangrey et al, 1969). Tests 6 and 14, as indicated, are close to or at the critical level of repeated loading and test 2 is clearly below it.

Anderson (1976) proposes a practical (graphical) method of predicting the cyclic behavior of a soil specimen subjected to varying cyclic stress levels, applied in one test. The equivalent cyclic shear strain at a second cyclic shear stress level, γ_{ce}, may be expressed by the equation:

$$\gamma_{ce} = \gamma_{c,N} + \Delta \gamma_{c,i} \tag{1}$$

where $\gamma_{c,N}$ is the cyclic strain accumulated at the first cyclic shear stress level in N cycles and $\Delta\gamma_{c,i}$ is the "immediate change in cyclic shear strain" assumed equal to the difference in shear strain in the first cycle (N = 1) of the two cyclic shear stress levels.

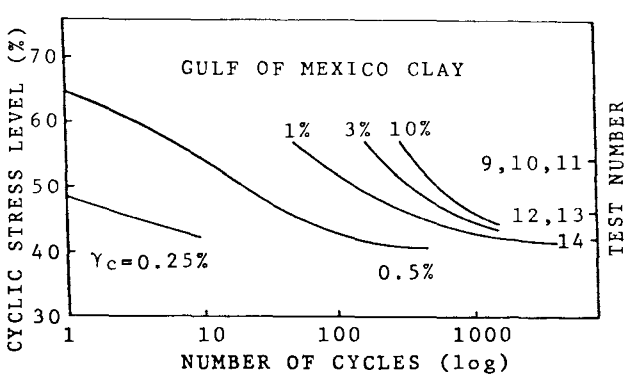

Fig.4 Cyclic Shear Strain versus Cyclic Stress Level and Number of Cycles

As an example, a Pacific Illite specimen is subjected to a cyclic shear stress of 51% (test 3) and starts at point A in Figure 3 for the first cycle (0.20 percent). Cycles 2 through 130 bring the cyclic shear strains to 0.50 percent (point B and $\gamma_{c,N}$ in Equation 1). The cyclic shear stress level is changed to 58 percent (test 5) and the 0.50 percent strain contour is followed up to point C. The shear strain in the first cycle of test 5 is 0.27 percent. The difference between the two first cycle strains of 0.07 percent ($\Delta\gamma_{c,i}$) is then added to yield 0.57 percent (point D and γ_{ce}). Cycle number 130 of this example test is therefore equivalent to cycle 70 in test 5 and the remainder of the test behaves similarly to test 5 from point D. Tests with two or more increasing or decreasing cyclic shear stress levels can be modelled in this way.

MODELLING CYCLIC BEHAVIOR BY PORE PRESSURE

The amount of degradation in a soil subjected to cyclic loading is indicated by the increase in cyclic strains or decrease in modulus. It is also indicated by the increase in excess pore pressures. The cyclic strains were different at equivalent points (B and D) in the two stress levels of the previous example (Figure 3), but the normalized pore pressures are approximately the same. Therefore, one can equate points at different cyclic stress levels directly by utilizing pore pressure (Seed, 1979). The normalized excess pore pressure at cycle 130 (same starting point and strain as the strain example) of test 3 is 0.256 (Figure 2). This same normalized excess pore pressure in test 5 corresponds to cycle number 72 which is very close to the graphical strain prediction. Table 3 shows the comparison of equivalent numbers of cycles predicted by the graphical strain approach (subscript a) and equal pore pressures (subscript b) for different tests at different strain levels. The column headings indicate which tests (cyclic stress levels) the predictions were made from and to.

The pairs of numbers agree well, but the number of equivalent cycles predicted by equal pore pressure is typically slightly higher than those predicted by strain. Other than scatter in the test results, this can be attributed to the fact that the "immediate change in cyclic shear strain" of the graphical strain approach is assumed to be equal to the difference between the first cycle strains (Anderson, 1976). As a cyclic test progresses, the modulus (secant) of the specimen decreases. A change in cyclic stress level at a lower modulus would yield a higher change in strain. This would make the equivalent strain and therefore the equivalent number of cycles higher (closer to the pore pressure predictions). It should also be noted that the last two columns for the Pacific Illite and the last three columns for the Gulf of Mexico clay are for tests coming from and going to similar stress levels and therefore the predicted values should be similar in these columns (which they are).

The pore pressure approach is essentially equivalent to drawing pore pressure contours in

Table 3 Comparison of Predicted Cycles

γ_c	PACIFIC ILLITE FROM/TO				GULF OF MEXICO CLAY FROM/TO			
(%)	7/3	7/5	3/5	8/5	14/9	12/10	13/9	13/11
0.25a	7	13	9	9	4	3	3	4
0.25b	10	8	4	9	3	3	1	1
0.50a	160	74	70	70	25	20	15	21
0.50b	170	82	72	85	22	29	20	14
0.75a	320	121	118	118	58	52	54	51
0.75b	343	133	121	131	66	72	50	54
1.00a	381	137	141	141	85	88	81	85
1.00b	393	151	145	152	102	115	89	104
1.50a	425	160	159	159	NA	162	130	144
1.50b	427	171	168	170	NA	140	122	144
3.00a	446	174	174	NA	NA	229	212	243
3.00b	454	185	181	NA	NA	235	213	253
10.0a	462	191	191	NA	NA	366	416	405
10.0b	500	199	192	NA	NA	345	385	430

Figures 3 and 4 and following these directly to different stress levels.

Stress plots (cyclic shear stress versus the change in applied effective vertical stress) can be drawn for these tests. As the cyclic test progresses, the effective vertical stress, $\bar{\sigma}_v$, decreases and the excess pore pressure, Δ_u, increases (the sum of these equals the constant total vertical stress). Figures 5 and 6 show the normalizedexcess pore pressures at different strains and cyclic shear stress levels for the Pacific Illite and Gulf of Mexico clays, respectively. Best-fit, straight line strain contours are drawn from the maximum normalized excess pore pressure of 0.8 (discussed previously) and through the corresponding data points. Both cyclic tests to failure and to cyclic equilibrium are plotted in these figures. The dashed line is a suggested boundary for the data. Tests with cyclic shear stress levels below this boundary cannot attain the strain levels of the contours in 3000 cycles. A general expression for normalized excess pore pressure as a function of cyclic stress level, τ_c, and cyclic strain for both clays can then be shown as:

$$\frac{\Delta_u}{\bar{\sigma}_{vo}} = 0.8 + m \cdot \tau_c \qquad (2)$$

where m is the slope of the strain contours in these figures. Figures 7 and 8 show the relationship between the slope, m, and the cyclic shear strain, γ_c for each clay.

To use the previous example, if the cyclic stress for test 3 (0.074 kg/cm^2) and the m-value for 0.50 percent shear strain (-7.3) are entered into Equation 2, a normalized excess pore pressure of 0.260 kg/cm^{2}is obtained. This compares to the actual value of 0.256 in

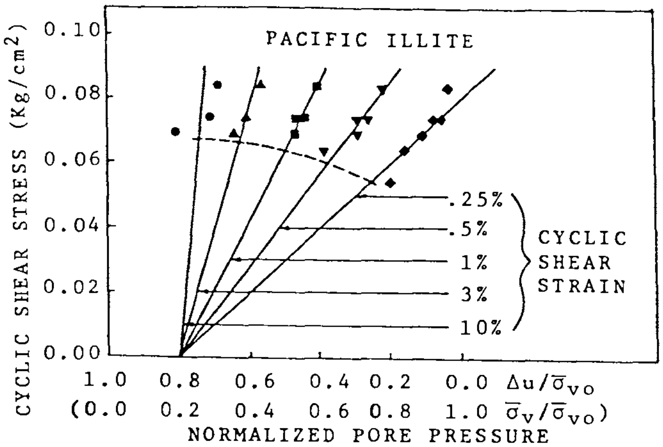

Fig.5 Cyclic Shear Stress versus Normalized Pore Pressure

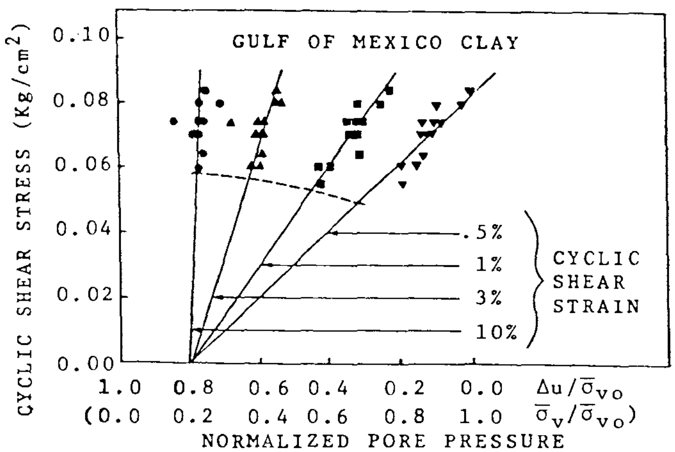

Fig.6 Cyclic Shear Stress versus Normalized Pore Pressure

test 3. If the normalized excess pore pressure remains the same, but the cyclic stress level is changed to 0.084 kg/cm^2 (test 5), a new m-value of -6.4 is obtained from Equation 2. This m-value corresponds to a cyclic shear strain of 0.60 percent in Figure 7 and is, in turn, equivalent to 75 cycles in test 5. This compares to an equivalent cyclic shear strain of 0.57 percent and 70 cycles by the graphical strain approach.

Decreasing modulus in a cyclic shear test is an indication of the progressive degradation in a soil specimen. The decreasing modulus can be normalized to the modulus in the first cycle of the test. Since these tests are stress-controlled, the cyclic shear stress level cancels in the normalized modulus and becomes simply a ratio of the shear strain in the first cycle to the shear strain in the Nth cycle ($\gamma_1/\gamma_{c,N}$). This strain ratio is plotted versus normalized excess pore pressure in Figures 9 and 10 for the Pacific Illite and Gulf of Mexico clays, respectively. All the tests fall on the same curve,

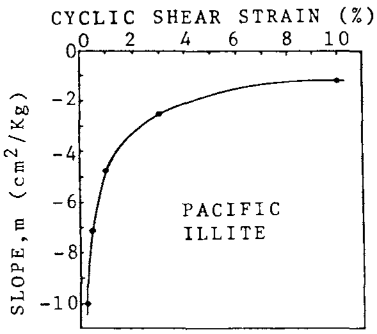

Fig.7 Slope, m, versus Cyclic Shear Strain

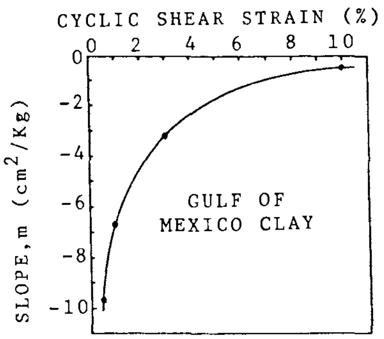

Fig.8 Slope, m, versus Cyclic Shear Strain

and this, therefore, is a unique relationship between excess pore pressure and strain. The curves are also valid for cyclic equilibrium tests as they merely progress down the first part of the curve and stop.

Figures 9 and 10 suggest that if equal pore pressures are used in going from one stress level to another, the strain ratio must remain the same. This is somewhat different from the graphical strain approach as equal values are added to γ_1 and $\gamma_{c,N}$, thus changing the ratio. From the previous example, a normalized pore pressure of 0.256 in test 3 corresponds to an equivalent strain of 0.64 percent and 81 cycles in test 5.

MODEL COMPARISON TO ACTUAL TESTS WITH VARYING CYCLIC STRESS LEVELS

Two tests with varying cyclic stress levels were performed on the Pacific Illite clay. In the first test, 500 cycles of a 37 percent cyclic shear stress level were applied to the specimen and then changed to 51 percent. This test continued 460 additional cycles to failure. Using test 3 as a model, the predicted number of cycles to failure was 450 and 435 for the graphical strain approach and the pore pressure approach, respectively. Test number 8 was not performed to failure so it cannot be used as a model for such, but the equivalent number of cycles in test 8 after the change in cyclic

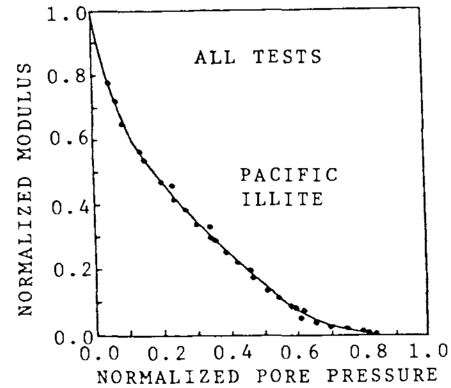

Fig.9 Normalized Modulus versus Normalized Pore Pressure

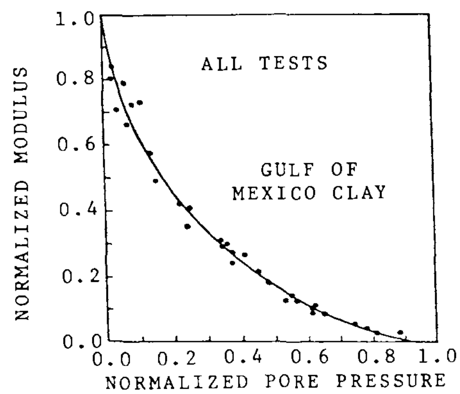

Fig.10 Normalized Modulus versus Normalized Pore Pressure

stress level was 40 cycles by the graphical strain approach and 39 cycles by the pore pressure approach. A similar test in which the cyclic stress level was changed to 58 percent was also performed. The actual test progressed about 100 cycles to failure. Using test 5 as a model, the predicted remaining cycles were 155 by the graphical strain approach and 150 by pore pressure.

CONCLUSIONS

Predicting the cyclic shear behavior of clays subjected to varying cyclic shear stress levels seems equally valid by either the graphical strain approach (Anderson, 1976) and the pore pressure method presented here. This is shown by the close agreement in predictions for the hypothetical tests in Table 3 and for the actual tests. However, the pore pressure method is easier and is recommended. This approach is similar to the suggestion made for sands by Seed (1979).

The prediction agreement and the unique pore pressure curves (Figures 9 and 10) indicate that pore pressure is a basic indication of the level of degradation in a soil specimen subjected to cyclic loading. If the progression of cyclic shear strain in a test is known (perhaps using data such as shown in Figures 3 and 4), the models presented in Figures 4 through 10 can be used to predict the pore pressure progression. The authors have found that the pore pressure modelling presented herein works equally well for 3 other clays and a cemented silty sand.

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