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SYNOPSIS Stress-controlled cyclic triaxial tests were performed on clay samples obtained from the subgrades of existing highway pavements in the State of Michigan. These samples were unconsolidated or isotropically consolidated and then tested under the following conditions:

1) confining pressures of 34.5 psi and 172.4 kPa (5 and 25 psi),
2) a range of stress ratio from 2 to 4,
3) a constant frequency of one cycle per second, and
4) at the natural water contents.

Test equipment included an MTS electrohydraulic closed-loop system which applies the sustained and cyclic loads to the sample in a stress-controlled mode, a minicomputer which controls the actuator of the MTS system to yield a loading function similar to that applied, by tandem axle truck, to the pavement section, and measuring devices including two vertical and two radial LVDT(s), a load cell, a pore water pressure transducer, recording devices and an ascilloscope.

The test results indicate that, for all samples, the plastic strain rate (in the logarithmic mode) assumes two values per test. The first of these values is constant from cycle number one to cycle number 100 while the second value is also constant and it controls the plastic strain beyond cycle number 100. This abrupt change in the strain rate values was observed in the unconsolidated and consolidated samples. Also, it was found that the ultimate stress ratio that it can be applied on the sample during cyclic loading is a function of the confining pressures and other sample parameters.

INTRODUCTION

In recent years the studies of soil behavior under repeated loading conditions were given considerable attentions particularly in the field of pavement design. This is so because researchers realized that the application of stresses to pavement materials by moving wheel loads is a transient one (4, 10, 2, 9, 15 and 6). To study the effect of cyclic loading on subgrade cohesive soils, stress-controlled cyclic triaxial tests were performed on clay samples obtained from the subgrade of existing highway pavements throughout the State of Michigan. These samples were unconsolidated or isotropically consolidated in a triaxial chamber and then they were subjected to undrained cyclic loadings under different testing conditions. These conditions included:

a) a range of cyclic stress ratio from 2 to 4, and
b) confining pressures of 5 and 25 psi.

In this paper the characteristics of the measured axial permanent deformations are discussed.

BACKGROUND INFORMATIONS

Timoshenko (17) credits Poncelet as being the first to consider the strength of materials under repeated loadings and to introduce the term "fatigue" to describe the resulting strength deterioration. Timoshenko (17) also credits Wohten for conducting the earliest and most extensive repeated load tests. Wohten found that the number of load cycles to failure increased as the cyclic stress intensity decreased. Other investigators (5,13, 14) studied the fundamental aspects of fatigue and developed hypotheses to explain the experimental data. These studies are still continuing and several theories are proposed each satisfying one or more aspects of the fatigue phenomenons and yet none being adequate for all cases (17,15,8).

Generally speaking, silt and clay subgrade materials (subjected to cyclic loading) will exhibit a stiffening behavior as the number of load applications increases (16,15,12). Each load application produces elastic (resilient) and plastic (permanent) deformations. The increment of plastic deformation of cohesive soils was found to be large during the first few load cycles. This increment decreases in subsequent load applications. After large number of load repetitions the change in permanent deformation per cycle becomes very small. The total permanent deformation of the test samples, however, increases with increasing number of cycles. Several models were used by different investigators to present the cumulative permanent strain (ε_p) and the number of load repetitions (N) data. The most popular of these, however, is the log log model (19) and the semi-log model (16,8,1,11). In this paper the former model will be used to study ε_p and N.

TESTING PROCEDURE AND EQUIPMENT

In this research program, all samples were trimmed to 5.4 cm (2.126 inches) in diameter and 13.6 cm (5.3543 inches) in length. The sample axial and radial deformations and the pore water pressure were measured using linear variable differential tranducers (LVDT). Two LVDT were mounted verically 180° apart and two horizontally, one at the middle of the sample and the other at one-third of the sample length from the base. The loading system consisted of a closed loop electro-hydraulic actuator. Most tests were conducted up to 30,000 cycles using a minicomputer modified sinusoidal wave form (shown in Figure 1) and a constant frequency of one hertz.

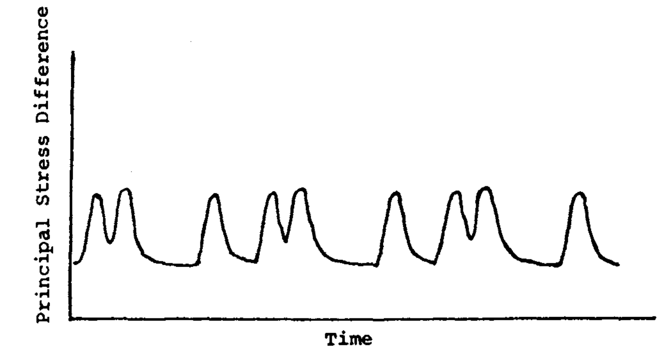

FIGURE 1. MINICOMPUTER MODIFIED SINUSOIDAL WAVE FORM

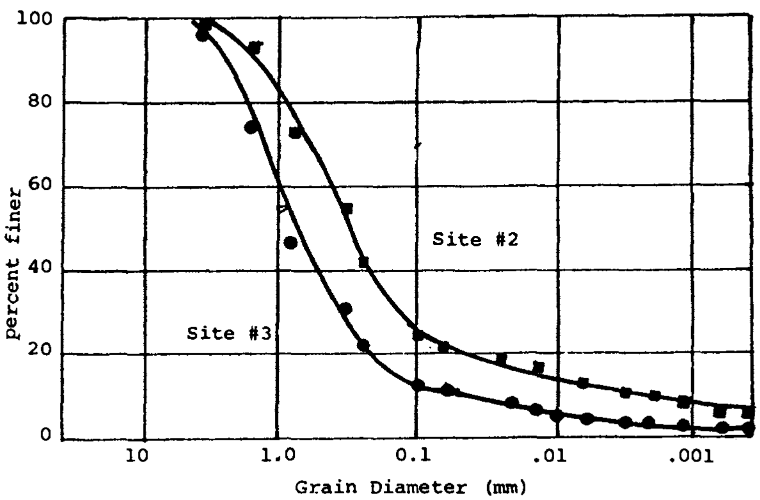

FIGURE 2 GRAIN SIZE DISTRIBUTION CURVES

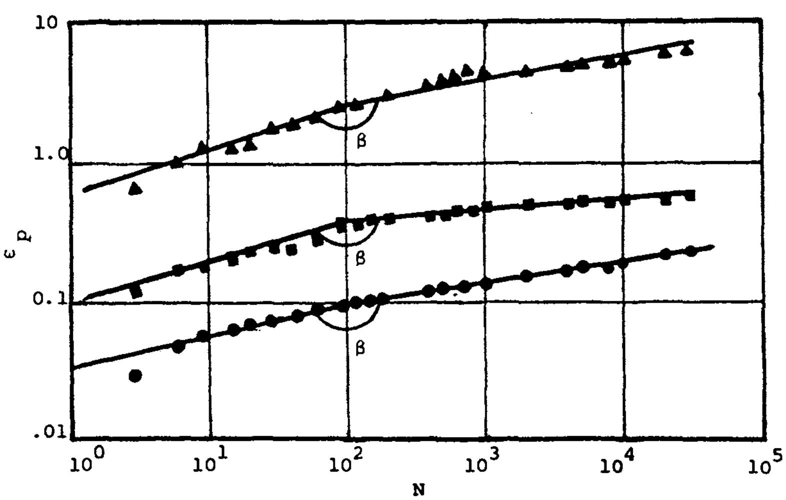

FIGURE 3. AXIAL PERMANENT STRAIN (ε_p) VERSUS NUMBER OF LOAD REPETITIONS (N) FOR SAMPLES CONSOLIDATED UNDER CONFINING PRESSURE OF 5 PSI, SITE #3.

TEST RESULTS

The grain size distribution curves of the test materials are shown in Figure 2. Typical cyclic triaxial test results are plotted in Figures 3 through 5. The permanent strains (ε_p) were calculated by dividing the average axial deformation (average of both longitudinal LVDT) by the original sample length. The samples were consolidated under a confining pressure of 34.5 and 172.4 kPa (5 and 25 psi) until the secondary compression curve was defined after which the repeated load test was commenced. The data in the figures are plotted in terms of the logarithm of ε_p against the logarithm of N. The straight lines in the figures were obtained using least square curve fitting technique. The two straight lines in each figure intersect at approximately cycle number 100. The angle of intersection is called the angle β in the figures.

DISCUSSION

Examination of Figures 3,4,5 and 6 indicates that the relationship between the cumulative permanent strain (ε_p) and the number of load repetitions (N) could be modeled using the following equation:

$$\log \varepsilon_p = \log a_i + S_i \log N \quad \text{OR}$$
$$\varepsilon_p = a_i N^{S_i} \qquad (1)$$

where: $\log a_i$ is the intercept in the figures,
S_i is the slope of segment i of the straight lines,
i = 1 or 2 relative to the proper segment of the straight line in the question.

It is apparent from the figures that the data points follow two distinctive straight lines. The first of these lines spans cycles number 1 through number 100, while the second line is for all cycles beyond cycle number 100. Similar data ,concerning the second straight line, were reported by several investigators (3, 19,16). They recommended that the first few cycles ,up to cycle number 100, be used for sample conditioning to seat the cap on the sample and to eliminate possible end imperfections.
Further studies of Figures 3,4,5 and 6 indicate that the angle of intersection (β) between the two straight lines

SYMBOL	$\frac{\sigma_1}{\sigma_3}$	Log a	β	δ_d	w	G_s
●	2	.00034	178	121	20.4%	2.72
■	3	.00108	171	121	21.2%	2.72
▲	4	.00598	170	121	20.0%	2.72

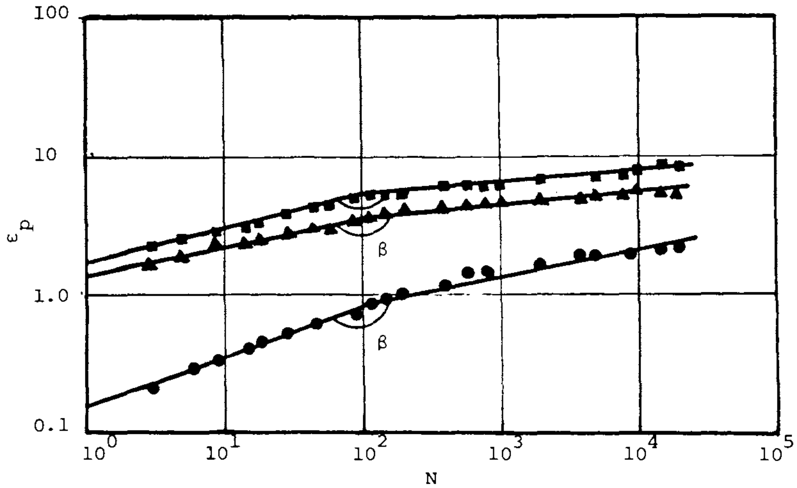

FIGURE 4. AXIAL PERMANENT STRAIN (ϵ_p) VERSUS NUMBER OF LOAD REPETIRIONS FOR SAMPLES CONSOLIDATED UNDER A CONFINING PRESSURE OF 25 PSI, SITE #3.

SYMBOLS	$\frac{\sigma_1}{\sigma_3}$	Log a	β	δ_d	w	G_s
●	2	.0015	175	121	19.8%	2.72
▲	2.5	.0135	172	120	20.6%	2.72
■	3	.0162	170.9	122	20.0%	2.72

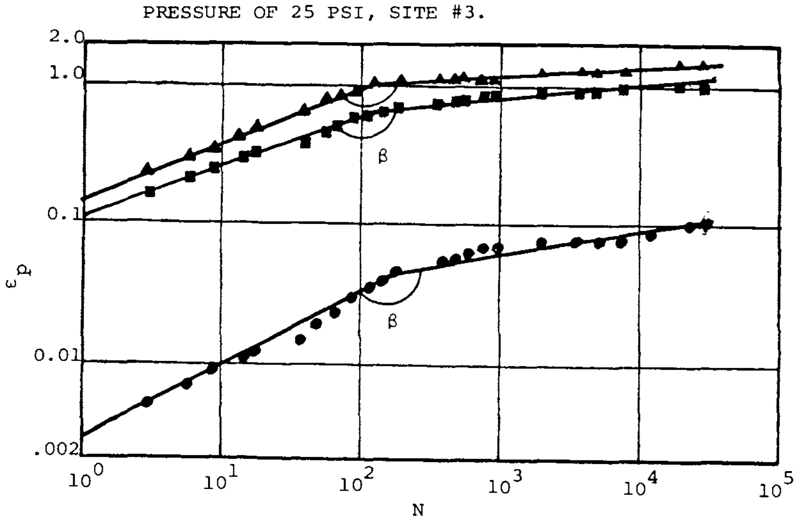

FIGURE 5. AXIAL PERMANENT STRAIN (ϵ_p) VERSUS NUMBER OF LOAD REPETIONS (N) FOR SAMPLES CONSOLIDATED UNDER CONFINING PRESSURE OF 5 PSI SITE #2

SYMBOLS	$\frac{\sigma_1}{\sigma_3}$	Log a	β	δ_d	w	G_s
●	2	.00028	168	118	22.4%	2.70
■	3	.00124	165	118	21.8%	2.70
▲	4	.0173	164	118	21.8%	2.70

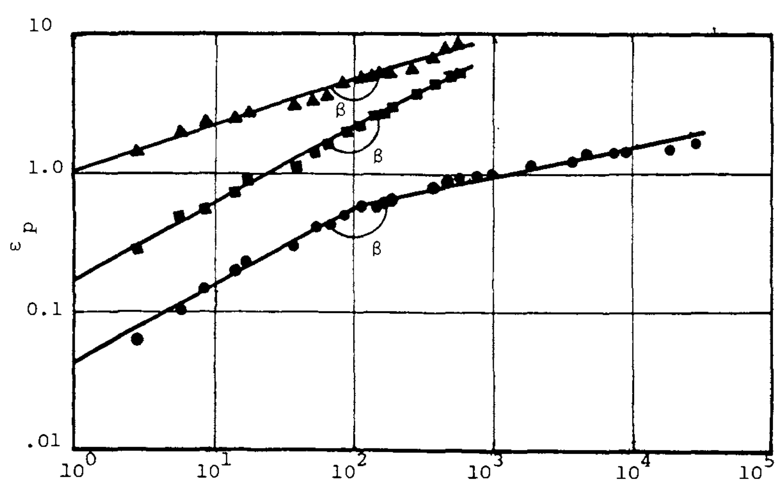

FIGURE 6. AXIAL PERMANENT STRAIN (ϵ_p) VERSUS NUMBER OF LOAD REPETITIONS (N) FOR SAMPLES CONSOLIDATED UNDER CONFINING PRESSURE OF 25 PSI, SITE #2.

SYMBOLS	$\frac{\sigma_1}{\sigma_3}$	Log a	β	δ_d	w	G_s
●	2	.00047	166	119	20.0%	2.70
■	2.5	.00173	179	118	19.8%	2.70
▲	3	.0108	180	118	19.8%	2.70

changes as the confining pressure and the cyclic stress ratio change. This may suggest that the sample behavior and the characteristics of permanent strain beyond cycle number 100 are influenced by the first 100 cycles and perhaps the first few cycles. This finding was expected because as the number of load repetitions increases the sample stiffness increases and thus the rate of plastic strain decreases. Consequently, if it is desired to study and compare the axial permanent strain of different samples or different soils, all specimens should be conditioned at the same stress level and to the same number of load cycles. Once the sample conditioning is reached the desired cyclic stress could be applied and the test could be commenced. The data through the conditioning cycles ,however, should be collected and reported regardless of its irregularities. Further, sample conditioning is a part of the sample stress history which may have substantial effects on the characteristics of the sample plastic strain.

As it was expected, the higher the cyclic stress difference the higher the permanent strain and for the same stress ratio the higher the confining pressure the higher the permanent strain. These results are shawn in Figures 7 and 8 and 9 respectively. It should be noted at this time that each data point in Figure 7 represents an independent sample tested to 30,000 load repetitions. Further examinations of Figures 3,4,5,6,8 and 9 indicate (not very conclusively) that the angle of intersection (β) increases as the stress ratio decreases. The ultimate value of this angle is 180° for a stress ratio of 1. For this case (if the sample is relatively undisturbed) the slope of the single straight line will be related to the compression index of the sample.

CONCLUSIONS

On the basis of the test results, the following conclusions were drawn:

a) all soil samples prepared for cyclic loading tests should not be subjected to any sample conditioning prior to testing. If sample conditioning is desired however, then all the test data (starting from cycle number one) should be collected, analysed and reported.
b) The cumulative plastic strain of clay samples is related to the number of load repetitions by an exponential function of the form of equation 1. The parameters of the equation are related to the sample and test variables.

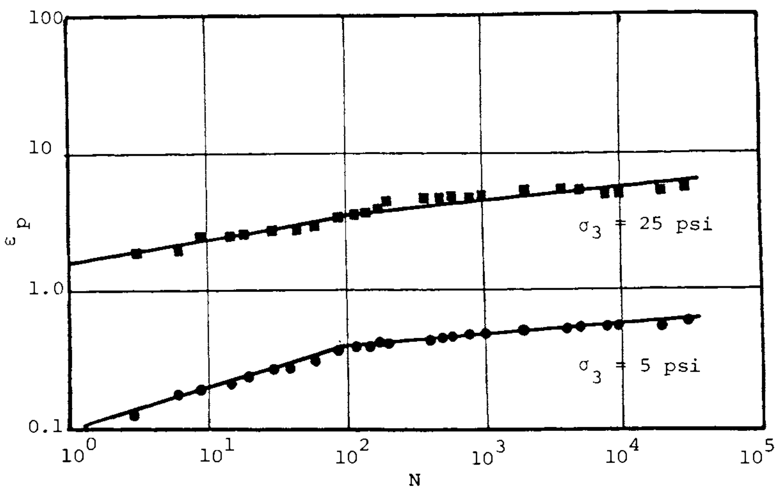

FIGURE 9. AXIAL PERMANENT STRAIN (ε_p) VERSUS NUMBER OF LOAD REPETITIONS (N) FOR σ_d/σ_3 = 2.0, SITE #3.

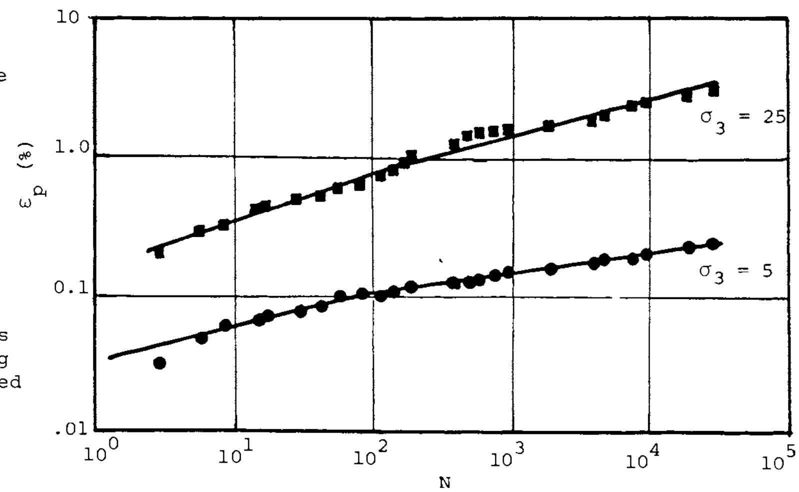

FIGURE 8. AXIAL PERMANENT STRAIN (ε_p) VERSUS NUMBER OF LOAD REPETITIONS (N) FOR σ_d/σ_3 = 1.0, SITE 3.

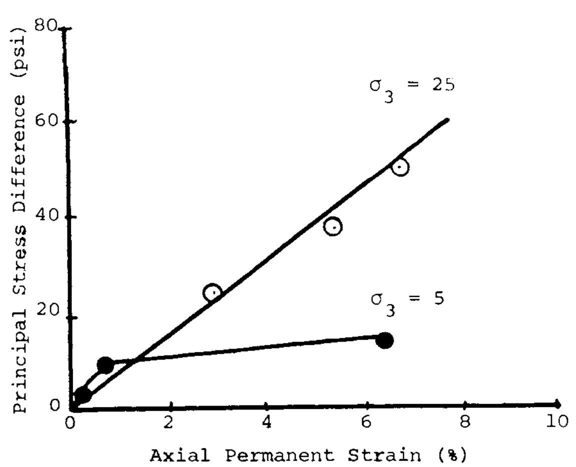

FIGURE 7. PRINCIPAL STRESS DIFFERENCE VERSUS AXIAL PERMANENT STRAIN.

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