Some Deformation Characteristics of A Lime-Stabilized Clay

PER E. FOSSBERG, Institute of Transportation and Traffic Engineering, University of California, Berkeley

A montmorillonitic clay was stabilized with 10 percent lime, molded at relatively high water contents, cured for 2 and 5 weeks, and tested with repeated loading in triaxial compression under various confining pressures and under drained and undrained conditions. The purpose of the test program was to investigate the effect of deviator stress and confining pressure on vertical strains and resilient modulus.

Each sample was tested at successively increasing deviator stresses. The results indicate that stress history effects have to be considered when evaluating permanent strains at low numbers of load repetitions, and also when interpreting such data in terms of fatigue behavior. Furthermore, the permanent deformations suffered under increasing repeated load stress levels depend largely upon the interplay between these stresses, the water content and the confining pressure; however, a general prediction as to the relative effect of these three factors cannot be made.

Not taking possible stress history effects into account, the tests show a relatively regular pattern of resilient strains, and conversely, resilient modulus, which invariably increases with increasing confining pressure and with decreasing deviator stress—a finding in full agreement with that obtained for other types of stabilized materials. Based on all the tests performed, reasonably good correlations exist between resilient modulus and principal stress ratio at various water contents. A convenient model for these relationships is presented.

•THE search towards rational methods in pavement design, based on elastic theory, has been intensified over the past few years. It appears that this work has mainly been concentrated in two areas: (a) development of adequate models for the theoretical analysis of layered systems, and (b) accumulation of knowledge of materials properties, particularly in regard to resilient behavior.

While model development has now advanced to a stage where multilayered systems can be analyzed fairly satisfactorily, knowledge of materials behavior necessary for the theoretical solutions is rather incomplete, particularly when dealing with stabilized materials. In the latter area, the present state of the art may be summarized as in Figure 1.

Seed et al (1) when investigating the effect of confining pressure, σ_3, and deviator stress, σ_d, on the resilient behavior of clays and clayey soils, found that the effect of confining pressure on the resilient modulus, M_R, is relatively small, while the magnitude of the applied deviator stress is all important (Fig. 1a).

Paper sponsored by Committee on Lime and Lime-Fly Ash Stabilization.

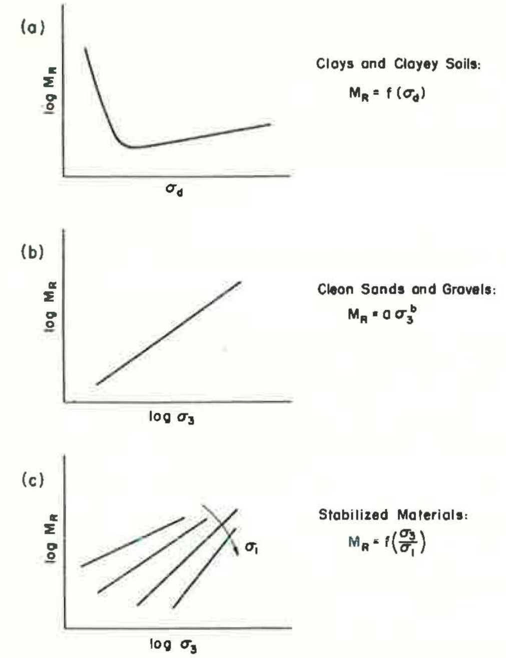


Figure 1. Resilient modulus as a function of stresses for three types of soils.

On the other hand, for cohesionless materials, as reported by Trollope et al (2) for a sand, and by Mitry (3) for a dry granular material, the resilient modulus increases with an increase in confining pressure, but is independent of the deviator stress level provided a failure condition is not reached. Mitry established a unique relationship between M_R and σ_3 (Fig. 1b).

For stablized materials having both frictional and cohesional characteristics, it is to be expected that both confining pressure and deviator stress play an important part. Thus, Gregg et al (4), testing a bitumen-stabilized windblown sand, found that M_R is markedly influenced by both confining pressure and deviator stress and is inversely related to the ratio between the major and minor principal stress. Monismith et al (5) reporting on an asphalt-emulsion-treated base arrived at essentially the same conclusion, that M_R increases with increasing confining pressure and with decreasing deviator stress. The general relationship (Fig. 1c) thus would hold true for these materials. While there is reason to believe that the same relationship holds true for cement-stabilized materials, the present paper serves to illustrate that it also applies to a lime-stabilized clay.

MATERIAL AND TEST PROCEDURES

The soil used in the present investigation was a black clay, residual from the weathering of norite, brought from Onderstepoort, Transvaal. The main mineral constituents are quartz and Ca-montmorillonite, with lesser amounts of hydrous mica, kaolinite and feldspars. Some of its relevant properties are given in Table 1. This clay was stabilized with 10 percent (by weight of dry soil) of high-calcium lime. This stabilizer content may seem high, but it is not when considering the large amount of clay present and the activity of this soil as compared to clayey gravels where a lime content of 4 to 5 percent is frequently used, but where the bulk of the material is coarse and relatively inert.

Samples to be tested in triaxial compression were prepared at two water contents, 51 and 77 percent. The clay-lime for the first set of samples was mixed for about one hour in a screw-type mixer operated under vacuum to obtain a high degree of saturation. In completion of the mixing, the samples were extruded through a 1½-in. diam. nozzle and cut in 3-in. lengths. The second set of samples was mixed for about an hour in a small pugmill, again with the application of suction. These samples were "cast" in 1½-in. diam. cylindrical molds, to a length of approximately 4 in., which was later trimmed down to 3 in. All

TABLE 1

INDICATOR PROPERTIES OF ONDERSTEPOORT CLAY

Property	Raw Soil	With 10 Percent Lime
Liquid limit, %	70	77
Plasticity index, %	42	28
Linear shrinkage, %	20	10
Material < 2 μ, %	57	
Specific gravity	2.69	
Activity	0.74	

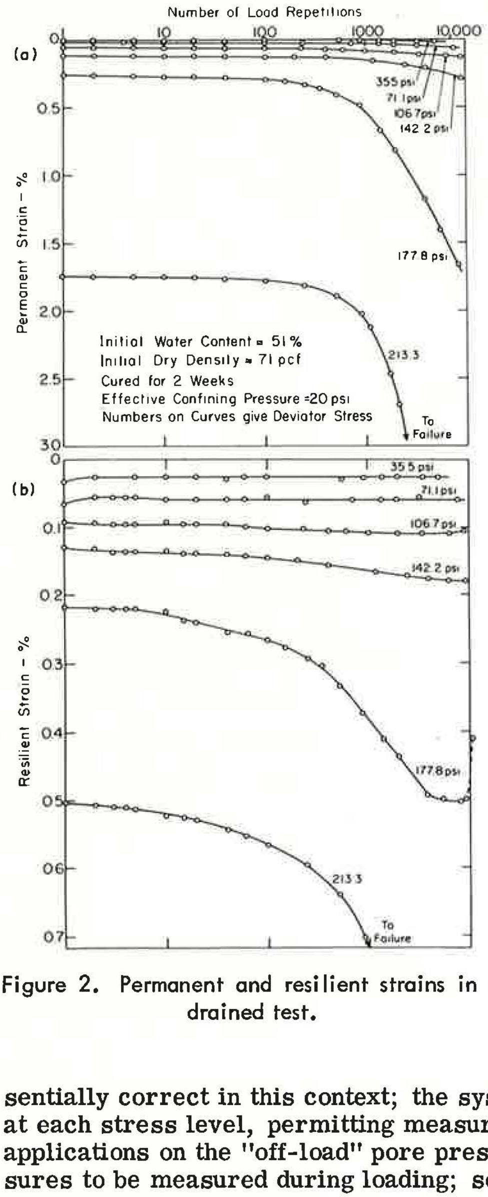


Figure 2. Permanent and resilient strains in a drained test.

samples were cured at 20 C, at a relative humidity exceeding 90 percent.

As it was desired to evaluate the test results in terms of effective stresses, full saturation of the samples was required. To achieve this, the samples upon completion of the curing period were soaked under water for four hours, with gradual application of suction at a rate of approximately 1 in. Hg per 10 min, up to 23 in. Hg, then the suction was gradually reduced over 30 min. The specimens were then mounted in triaxial cells, and isotropically consolidated overnight at pressures of 10, 20, and 40 psi. Prior to repeated load testing, an internal back pressure was gradually applied up to 30 psi, while simultaneously the cell pressures were increased to 40, 50, and 70 psi, respectively.

The samples were tested in repeated loading, using equipment basically similar to that used at the Soil Mechanics and Bituminous Materials Laboratory of the University of California, Berkeley, described by Seed et al (6). The frequency of loading was 20 per min, with a load duration of $^3/_4$ sec. Different samples were used for each confining pressure, but each sample was tested at increasing stress levels until failure. The number of stress applications at each stress level generally ranged between 5000 and 9000. Vertical deformations were measured with dial gages.

The specimens were tested under drained or undrained conditions, in the latter case with measurement of pore pressures. The term undrained is not essentially correct in this context; the system was closed during the whole test sequence at each stress level, permitting measurement to be made of the effect of repeated load applications on the "off-load" pore pressure. The equipment did not allow for pore pressures to be measured during loading; so during each stress application, temporary "drainage" took place.

TEST RESULTS

Typical patterns of permanent and resilient strains as functions of number of load repetitions at increasing stress levels are shown in Figures 2 and 3. Figure 2 represents drained test conditions at an effective confining pressure of 20 psi, for the clay-lime mixture with an initial water content of 51 percent, cured for 2 weeks prior to repeated loading. Figure 3 gives data for undrained tests on the clay-lime with an initial water content of 77 percent cured for 5 weeks, and also the variation in off-load pore pressures, starting with an effective confining pressure of 10 psi.

Permanent Strains

Each sample having been tested at increasing stress levels, the permanent strain data have been plotted in terms of cumulative values (Figs. 2a and 3a). Typically, the

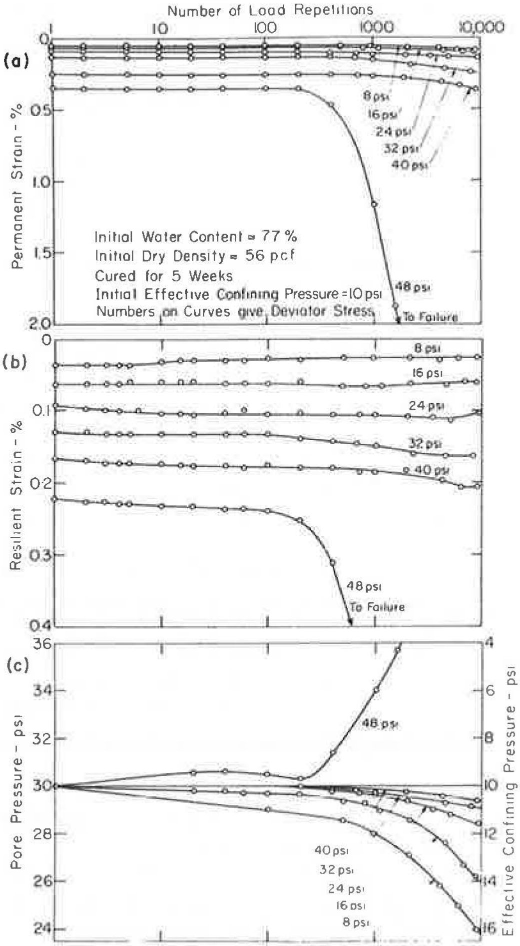


Figure 3. Permanent and resilient strains and pore pressures in an undrained test.

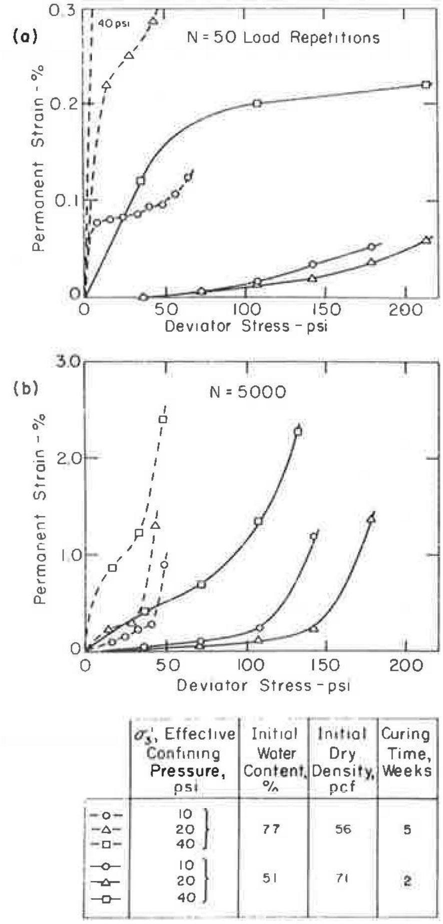


Figure 4. Permanent strains vs deviator stress in drained tests at various confining pressures.

curves for permanent strain show only little increase with the number of load repetitions, N, at low counts. Invariably in the present investigation, however, the slope of permanent deformation versus the logarithm of the number of load applications in this low range of N appears to be even flatter than normal, representing approximately a constant value. Part of this effect may probably be ascribed to stress history. Beyond this range of N (usually occurring around 100), the permanent strains increase markedly, indicating that the effect of stress history is less significant. This would tend to show the desirability of carrying the number of load repetitions well beyond 100 when doing tests at increasing stress levels on the same sample. On the other hand, the number of load repetitions applied at each but the last stress level was not sufficient to show whether the sample was perfectly stable (which would be indicated by the permanent strains tending towards a constant value at high numbers of load repetitions), or whether a failure condition was approached (which would be indicated by a drastic increase in permanent strains at high counts). Evaluation of fatigue life at various stress levels was not the purpose of this investigation; however, it clearly is important that such stress history effects be taken into account when interpreting fatigue test data.

Figure 4 gives the available data for permanent strains as a function of deviator stress and confining pressure under drained conditions for the two clay-limes. The

values are cumulative in the sense that Figure 4a sums up the permanent strains suffered during the first 50 load applications at each stress level, disregarding deformations occurring thereafter; in Figure 4b the same is true at 5000 load applications. The slope of the curves thus represents the rate at which permanent strains increase with deviator stress.

For the clay-lime with an initial water content of 51 percent, the tests carried out at 10-psi and 20-psi confining pressure show a progressive increase in permanent strains with increasing deviator stress, indicating a gradual weakening of the sample—the effect of which is more pronounced the lower the confining pressure. In all the other samples, however, permanent strains increase relatively rapidly during the application of the lowest deviator stresses. The reason for this effect, which is more pronounced the higher the confining pressure, is not clear, but it would appear that consolidation of the sample cannot be the cause as the observed drainage (or, at the lower confining pressures, water absorption) during this first phase was relatively small. This conclusion is corroborated by the insignificant change in pore pressures that took place in the undrained samples during this first phase. In the second phase, the permanent strains again increase progressively with the deviator stress.

The data in Figure 4 show that the effect of confining pressure on permanent strains cannot always be predicted, particularly when dealing with materials at high water contents or relatively low dry densities. The aspect of fatigue behavior, being highly related to imposed strains, also cannot be uniquely related to confining pressures.

Pore Pressures

Figure 3c shows a typical pore pressure development in an undrained test under an initial confining pressure of 10 psi. At low and intermediate levels of deviator stress, repeated stress applications are accompanied by a decrease in off-load pore pressures, indicating dilatancy of the soil skeleton. Other tests in this series, carried out at an initially high confining pressure (40 psi), consistently show an increase in pore pressures (serving to reduce the effective confining pressure) indicative of compression

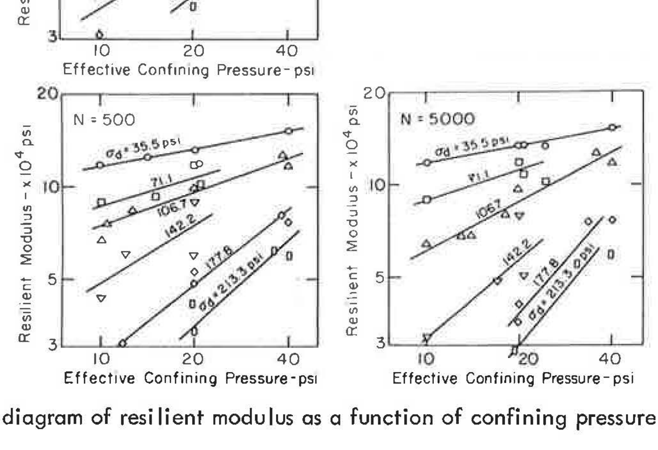


Figure 5. Typical diagram of resilient modulus as a function of confining pressure and deviator stress.

of the soil skeleton during repeated loading. These findings would indicate that the concept of a critical relationship between void ratio and confining pressure may, at least qualitatively, be valid also for stabilized soils.

At high levels of deviator stresses, a tendency towards compression is indicated (and more so, the higher the initial confining pressure) in that the pore water pressure increases, thereby reducing the effective confining pressure. This effect of reduced confining pressure is accompanied by rapidly increasing permanent and resilient strains indicative of incipient failure.

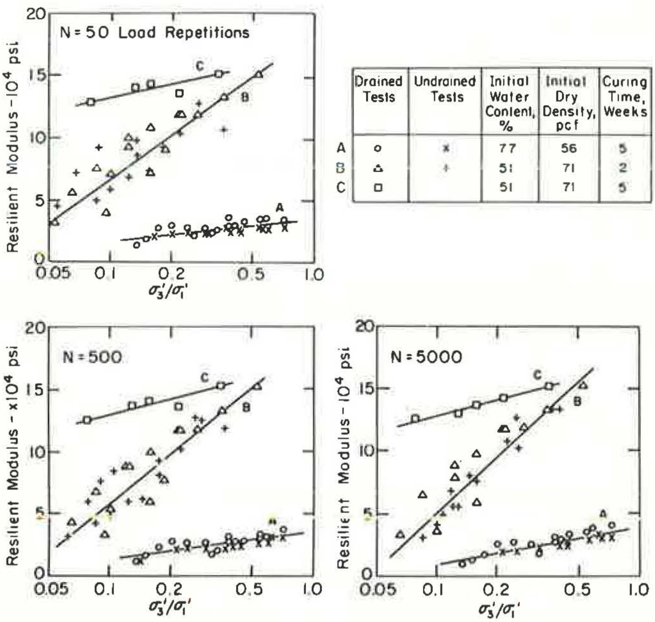
In this context an opposite effect observed in undrained tests on an asphalt-stabilized sand (7) is of interest. At fairly low initial confining pressures, but even at stress levels close to failure, pore pressures were found to fall with increasing number of load applications, which would indicate that at this confining pressure the void ratio of the sample was below the "critical." In this dilatant state close to failure, a remarkable reduction in resilient strains took place, probably due to the increased effective confining pressure.

Resilient Strains

In Figures 2b and 3b, resilient strains increase progressively with increasing deviator stress. At low and intermediate stress levels, there is relatively little variation of resilient strain with number of load repetitions. Whether this behavior is a typical feature of this material or is due to stress history effects has not been established. In accordance with expectations, a comparison of test results obtained at various confining pressures shows that resilient strains decrease with increasing effective confining pressure.

Resilient Modulus

The resilient modulus (deviator stress divided by resilient strain) increases with increasing confining pressure and with decreasing deviator stress. This is well illustrated in Figure 5, which gives data for the clay-lime mixture with an initial water content of 51 percent and cured 2 weeks prior to repeated loading. In accordance with



	Drained Tests	Undrained Tests	Initial Water Content, %	Initial Dry Density, pcf	Curing Time, Weeks
A	o	x	77	56	5
B	Δ	+	51	71	2
C	□		51	71	5

Figure 6. Model for the relationship between resilient modulus and principal stress ratio.

the findings for resilient strains, there is relatively little variation in resilient modulus with number of stress applications, except at high deviator stresses where the resilient modulus is markedly reduced at high numbers of load repetitions.

An analogous diagram to that in Figure 5, although somewhat less consistent, is obtained for the clay-lime mixture molded at 77 percent water content. The data in Figure 5 do not strictly comply with the concept of the resilient modulus being a simple function of the principal stress ratio, but do indicate that such a relationship is a reasonable approximation. Various models can be devised to establish the desired relationship; the best representation found is given in Figure 6, based on data from both drained and undrained tests and involving the major and minor principal effective stresses, σ_1' and σ_3', respectively. This semilogarithmic representation yields a relatively small spread of the data points and allows for a linear relationship to be drawn. Thus, for the clay-lime molded at a water content of 51 percent and cured for 2 weèks, this line can, at 5000 load repetitions, reasonably well be expressed by

$$M_R = 200.000 \; (1 + 0.75 \log \sigma_3'/\sigma_1') \text{ psi}$$

at least within the range of data here available.

For the same clay-lime mixture, but cured for 5 weeks, a limited amount of data based on undrained tests, were produced. As shown in Figure 6, the increased curing time causes an increase in the resilient modulus but it appears that the modulus for this longer cured sample is less affected by the principal stress ratio. This is reasonable, in that the cohesional component of resistance, being essentially independent of confining pressure, is proportionately larger the longer the curing time. In support of this, reference can be made to shear strength tests on this clay-lime as reported by Fossberg (8), showing that the lime has an immediate effect in increasing the true angle of internal friction, but that long-term gain in strength is mainly achieved through increase in true cohesion.

Figure 6 also gives the resilient modulus of the clay-lime mixture molded at a water content of 77 percent and cured for 5 weeks. Again we note the practically linear correlation with the logarithm of the principal stress ratio. As is to be expected, the resilient modulus at any one stress condition is considerably lower for this material than for the same molded at 51 percent water content.

CONCLUSIONS

The present investigation has shown that the resilient properties of a lime-stabilized clay, possessing pronounced cohesional and frictional characteristics, are strongly influenced by both deviator stress and confining pressure. This is in full agreement with the reported behavior of asphalt-stabilized sands and asphalt-treated granular bases. Furthermore, similar behavior can be expected for cement-stabilized materials, as also for natural "intermediate" soils, such as certain silts and clayey gravels.

A reasonable relationship has been established between resilient modulus and principal stress ratio at various water contents. If the same kind of relationship can be obtained for other types of stabilized materials and for intermediate soils, the full range of combinations of cohesional and frictional properties will be covered.

ACKNOWLEDGMENTS

The investigations described in this paper were conducted in 1964, when the author was employed by the National Institute for Road Research, Pretoria, South Africa. The laboratory tests were carried out with great skill and patience by Eric David.

REFERENCES

1. Seed, H. B., Chan, C. K., and Lee, C. E. Resilience Characteristics of Subgrade Soils and Their Relation to Fatigue Failure in Asphalt Pavements. Proc., Conf. on Structural Design of Asphalt Pavements, Ann Arbor, Mich., 1962.

2. Trollope, D. H., Lee, I. K., and Morris, J. Stresses and Deformations in Two-Layer Pavement Structures Under Slow Repeated Loading. Proc., Australian Road Research Board, Vol. I, Part II, 1962.
3. Mitry, F. C. Determination of the Modulus of Resilient Deformation of Untreated Basecourse Materials. Ph.D. Thesis, University of California, Berkeley, 1964.
4. Gregg, J. S., Dehlen, G. L., and Rigden, P. J. On the Properties, Behavior and Design of Bituminous Stabilized Sand Bases. Proc., Conf. on Structural Design of Asphalt Pavements, Ann Arbor, Mich., 1967.
5. Monismith, C. L., Terrel, R. L., and Chan, C. K. Load Transmission Characteristics of Asphalt-Treated Base Courses. Proc., Conf. on Structural Design of Asphalt Pavements, Ann Arbor, Mich., 1967.
6. Seed, H. B. and Fead, J. W. N. Apparatus for Repeated Load Tests on Soils. ASTM Spec. Tech. Publ. No. 254, 1959.
7. Fossberg, P. E. Unpublished Test Results. National Institute for Road Research, Pretoria, South Africa, 1964.
8. Fossberg, P. E. Some Fundamental Engineering Properties of a Lime-Stabilized Clay. Proc., Sixth International Conf. on Soil Mechanics and Foundation Engineering, Montreal, Canada, Vol. I, p. 221-225, 1965.