Stress-Strain Characteristics of Compacted Clay Under Varied Rates of Strain

J. O. OSTERBERG AND WILLIAM H. PERLOFF, JR.,
Respectively, Professor of Civil Engineering and Graduate Research Assistant, Technological Institute, Northwestern University, Evanston, Ill.

• THIS PAPER is to be considered as a progress report on the study of the effect of rate of loading on stress-strain properties of a compacted clay. The idea for the study initiated from failure of a 40-ft fill in Illinois that could not be accounted for by the usual stability analysis. The fill, placed on a natural ground of many times the compressive strength of the embankment, consisted of a silty brown clay. The clay is an oxidized and weathered glacial till. Its compaction and index properties are shown in Figure 1.

The embankment was placed late in the fall when it was difficult to obtain drying. Many tests on borings taken the following spring indicate that on the average the fill was placed 2 percent wetter than standard AASHO optimum, but many individual tests showed water contents as much as 8 percent over optimum. The dry density, however, met the specified 95 percent compaction. The exact nature of the failure is not known, inasmuch as remedial measures were taken before complete measurements of the failed embankment slope and rates of movements could be made. The top of the embankment, however, settled a few feet and outward movement occurred at the lower part of the slope. To meet the time schedule for placing the concrete pavement, the failure was remedied by removing the entire failed portion plus additional soil and replacing with suitably compacted fill.

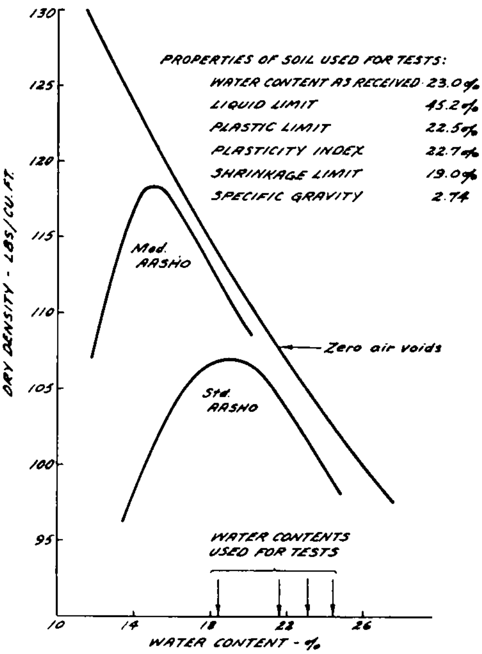

Figure 1. Compaction and index properties of soil used for tests.

The factor of safety for a circular arc failure through the embankment without including the much stronger embankment foundation, based on the average shear strength obtained from unconfined compression tests, was found to be in excess of 2.5. Even with the smallest single test value, the factor of safety was greater than 1.3. Therefore, either the failure was not of the circular arc type or the strength tests did not represent the soil strength *in situ*. It was this apparent discrepancy which engendered the interest in securing soil from the embankment for more detailed study. It was not the immediate intention to try to explain the failure, but to study more fully the effect of changing water contents and rates of strain on the stress-strain properties of the soil as a possible clue to a more complete explanation.

PREPARATION OF SPECIMENS

All specimens were prepared from a 200-lb sample taken from the embankment fill when it was removed for replacement. No soil for any test was reused and none of the soil was allowed to dry below 18 percent water content. All specimens at a given water content were prepared at the same time by running a sufficient quantity of soil through a meat grinder a minimum of three times. If a moisture content lower than the natural moisture content was desired, the soil was allowed to dry slowly between grindings in a humid room. For higher moisture contents the clay was sprayed with a fine mist between grindings. Following thorough mixing, the soil was stored for at least two weeks to allow migration to insure uniform distribution of moisture throughout the sample. Storage was obtained under 100 percent humidity by enclosing the soil in a polyethylene bag and placing it on a platform in a sealed jar with water in the bottom.

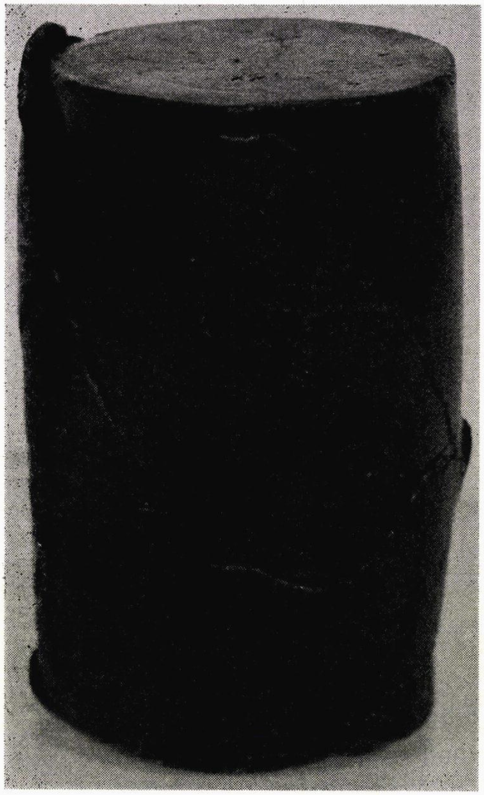

Figure 2. Typical sample at failure.

Specimens from the cured soil samples were prepared with a Harvard miniature compactor (*1*), by compacting in four layers using 25 tamps per layer. Unfortunately it was not possible to compact with the same force on the compacting rod for all water contents, for at the low water contents a larger force was needed to break up the lumps and at the higher water contents a smaller force was needed to avoid pushing the soil away from the rod. Compaction pressures varied from 90 psi at 24.4 percent moisture content to 300 psi at 18.5 percent moisture content.

After ejection from the cylindrical mold, each specimen was weighed and enclosed in a polyethylene cover, which in turn was covered by a rubber membrane. All specimens were allowed to cure in a humid room for two to three weeks, so test results for individual specimens at the same water content would not be affected by age hardening if tested a few days apart.

METHOD OF TEST

All specimens were tested in a triaxial compression device at zero lateral pressure at varying rates of strain. To maintain virtually constant water content throughout a test, the protective coverings were left on the specimens and the lower portion of the triaxial chamber was filled with water so that there was 100 percent humidity in the upper portion. Piston friction in the triaxial apparatus was negligible due to a ball bushing sleeve around the piston. Specimens were completely sealed, allowing no drainage throughout all tests. Varying rates of loading were applied by means of a variable-speed drive and several gear reductions permitting continuous rates of loading from 0.05 to 0.0001 in. per minute. The total test time varied from 10 min to 4 days. Load and strain dial readings were taken at sufficient intervals to plot complete stress-strain curves. Load and strain dial readings were trans-

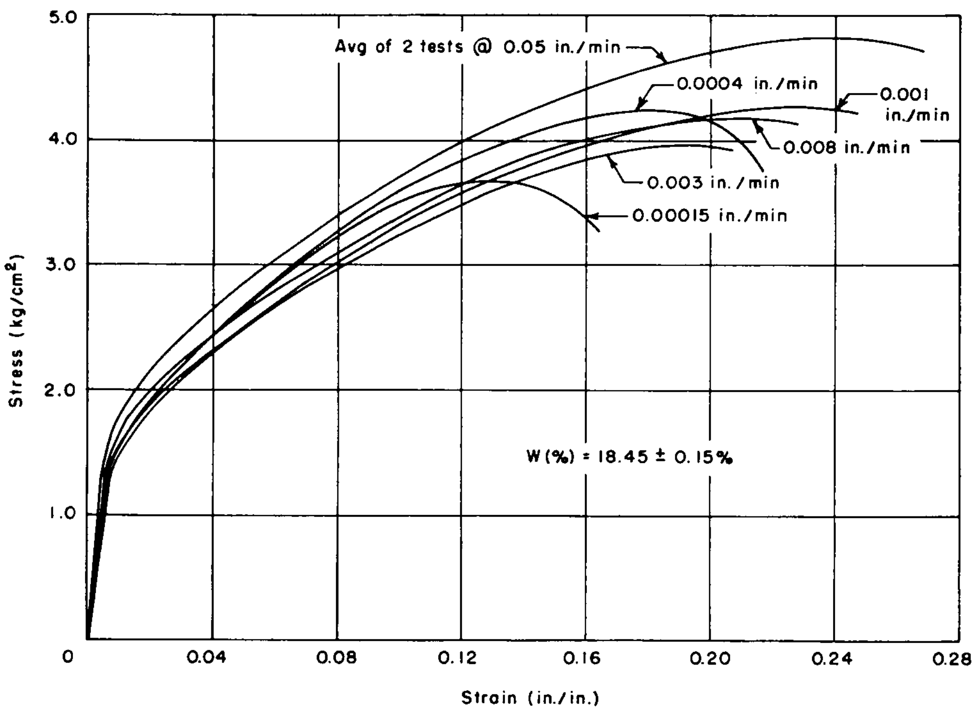

Figure 3. Composite of unconfined compression test results; water content, 18.45 ± 0.15 percent.

formed to stress-strain data by an electronic computer using a specially written program. Computation time was reduced to less than 10 percent of that required for hand calculation.

The failure pattern shown in Figure 2 is typical of about 70 percent of the failures. Some failures were barrel shaped and others showed many parallel shear planes, but all showed some shear cracks. The types of failure had no apparent relationship to rate of strain or water content.

TEST RESULTS

Figures 3, 4, 5 and 6 indicate the test results for water contents of 18.5, 21.6, 23.1, and 24.4 percent, respectively. It is seen from these curves that the shapes for all water contents are quite similar, but the magnitude of maximum stress and the modulus of deformation vary with water content. All the stress-strain curves are straight in the lower portion, then bend at a more or less definite point to a curved and much flatter portion until the maximum stress is reached. The maximum occurs at strains varying from 7 to 26 percent, with the maximum stress occurring at smaller and smaller strains as the rate of loading is decreased.

The relationship between maximum stress (unconfined compression strength) and rate of strain is shown in Figure 7. Although considerable scatter in the test points is evident, especially for the lowest water content, there is a definite indication of lower strength for decreasing rates of strain. Each straight line was fitted by the method of least squares. The percent reduction in strength with decreasing rates of strain is greater for the higher water contents. In fact, the strength reduction is almost 50 percent for the higher water contents for the range of strain rates tested (test time, 10 min to 4 days).

Figures 8, 9, 10 and 11 show the lower portion of the stress-strain curves to an

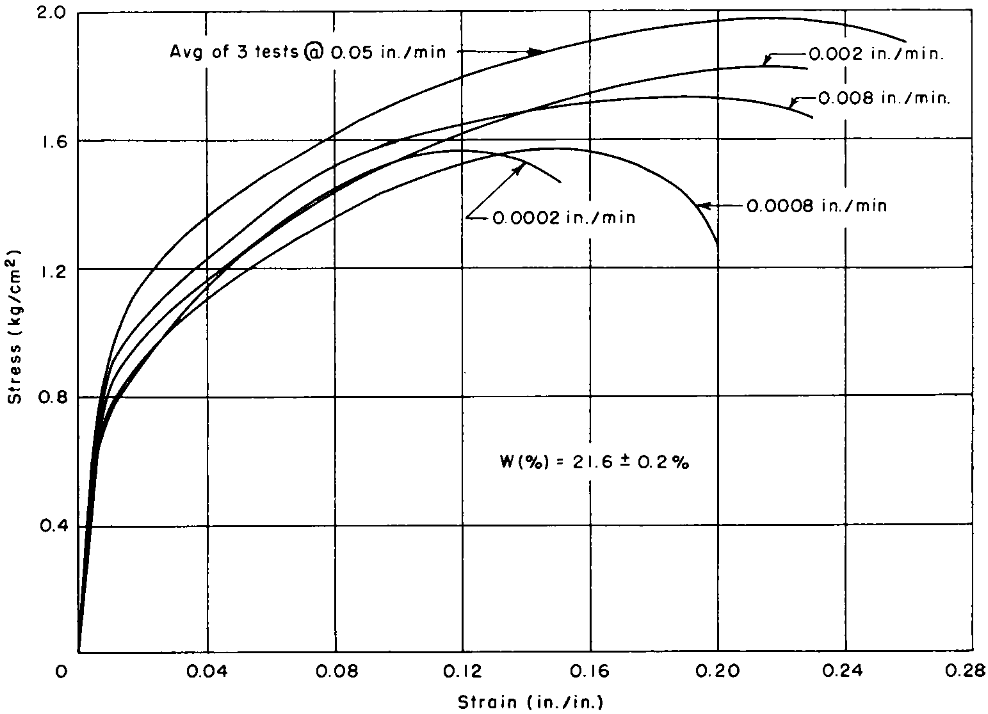

Figure 4. Composite of unconfined compression test results; water content, 21.6 ± 0.2 percent.

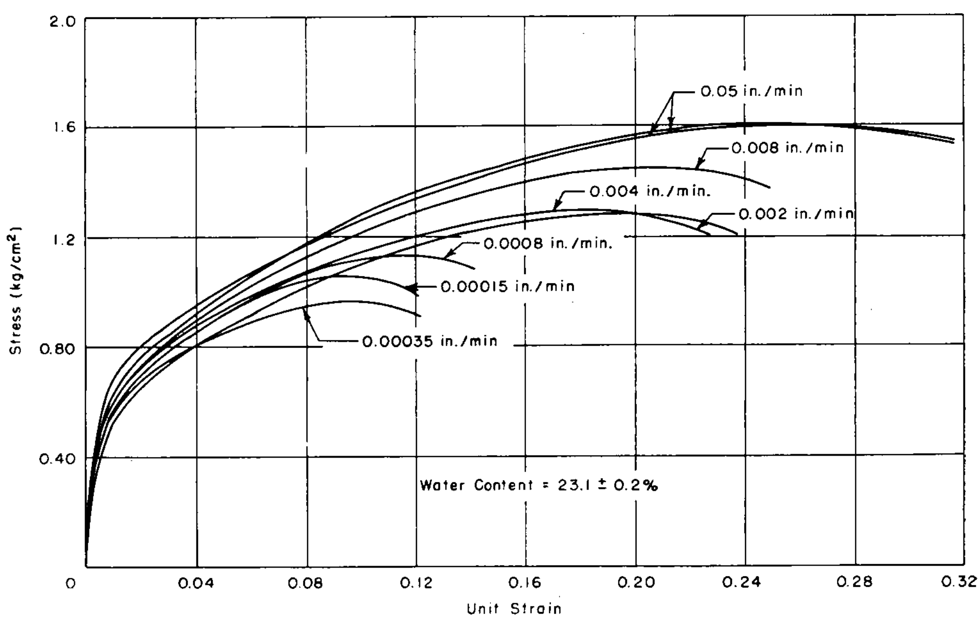

Figure 5. Composite of unconfined compression test results; water content, 23.1 ± 0.2 percent.

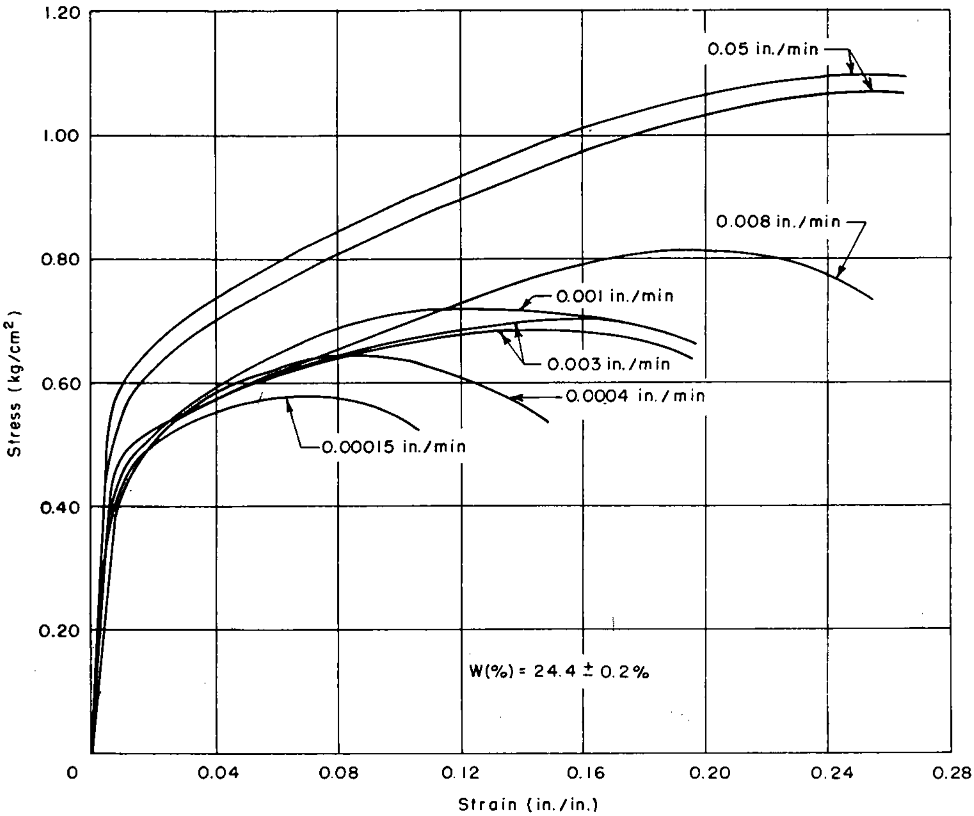

Figure 6. Composite of unconfined compression test results; water content, 24.4 ± 0.2 percent.

enlarged scale, each for a different water content. It is seen that in every case the initial portion is a straight line whose slope is independent of the rate of strain at a constant water content. The curves were corrected to pass through the origin when necessary. A typical set of uncorrected curves is shown in Figure 12. An attempt was made to determine a relationship between the limit of proportionality and rate of strain, but no consistent relationship was apparent. However, if the point of maximum curvature, here called "yield point," just beyond the proportional limit is used, the relationship between yield point stress and rate of strain is that shown in Figure 13. Here again, the reduction in yield point stress with decreasing rates of strain is apparent.

The relationship of unconfined strength to water content for varying rates of strain is shown in Figure 14. In general, for a given rate of strain the strength decreases as the water content increases; at decreasing rates of strain the curves are displaced in the direction of lower strength. Figure 15 shows a similar relationship between yield point stress and water content.

Figure 16 shows the relationship between the water content and the modulus of deformation. A remarkably straight line is obtained at water contents above the shrinkage limit. It should be pointed out that in comparing results at different

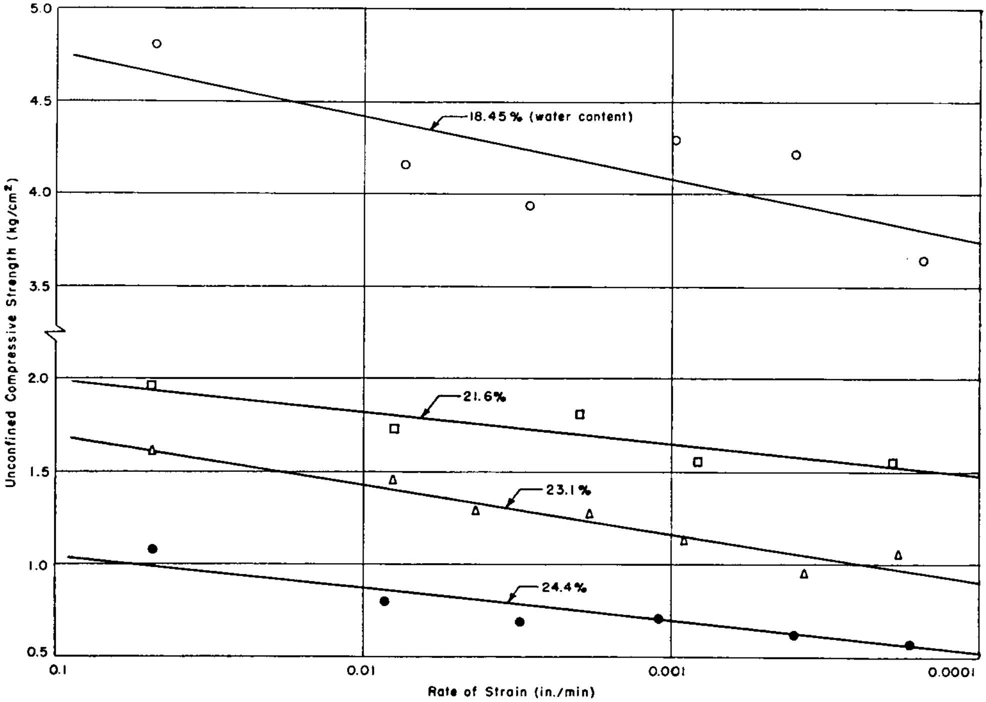

Figure 7. Relationship between rate of strain and unconfined compressive strength.

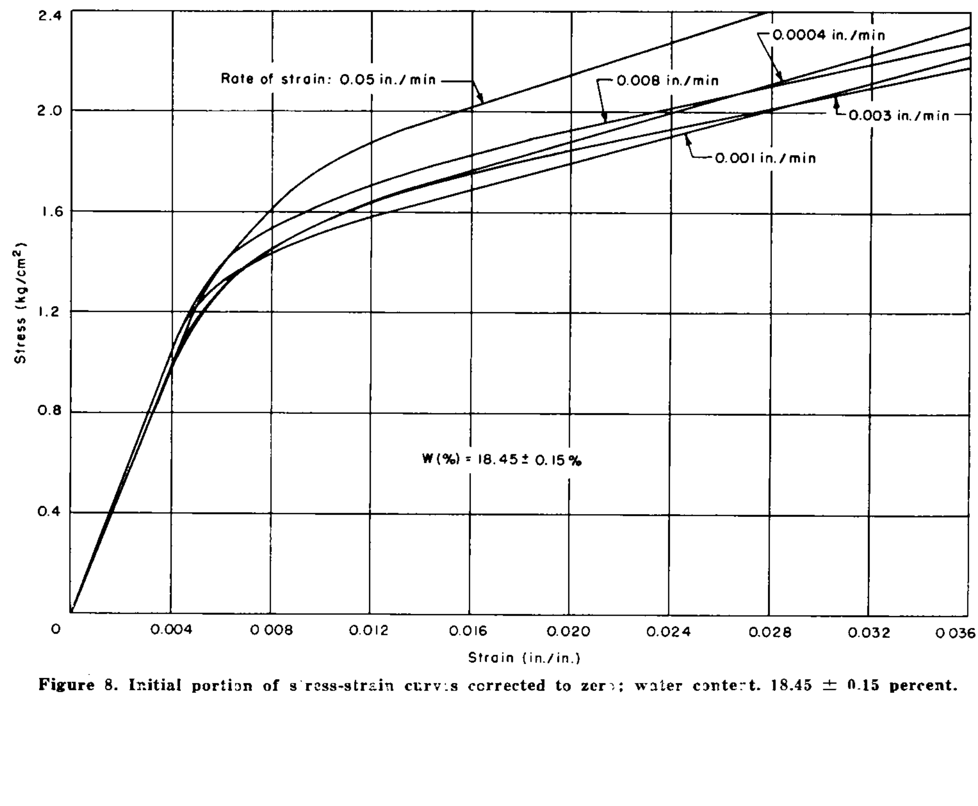

Figure 8. Initial portion of stress-strain curves corrected to zero; water content. 18.45 ± 0.15 percent.

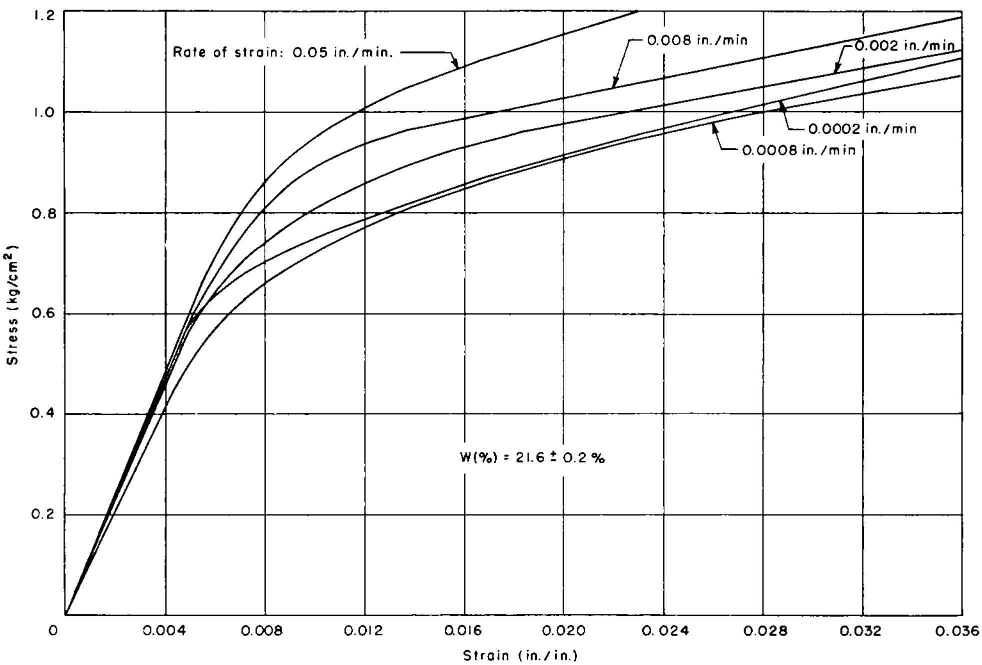

Figure 9. Initial portion of stress-strain curves corrected to zero; water content, 21.6 ± 2.0 percent.

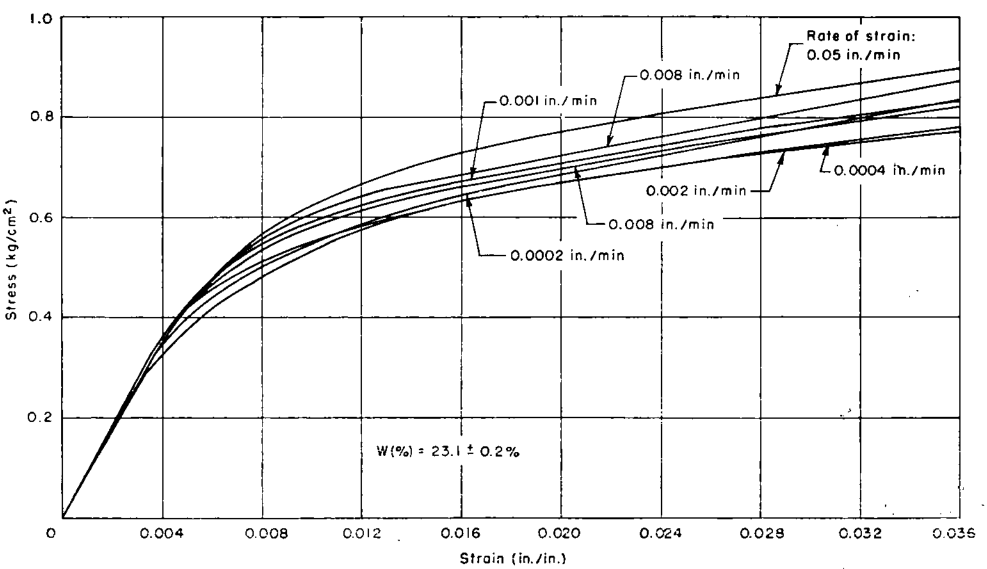

Figure 10. Initial portion of stress-strain curves corrected to zero; water content, 23.1 ± 0.2 percent.

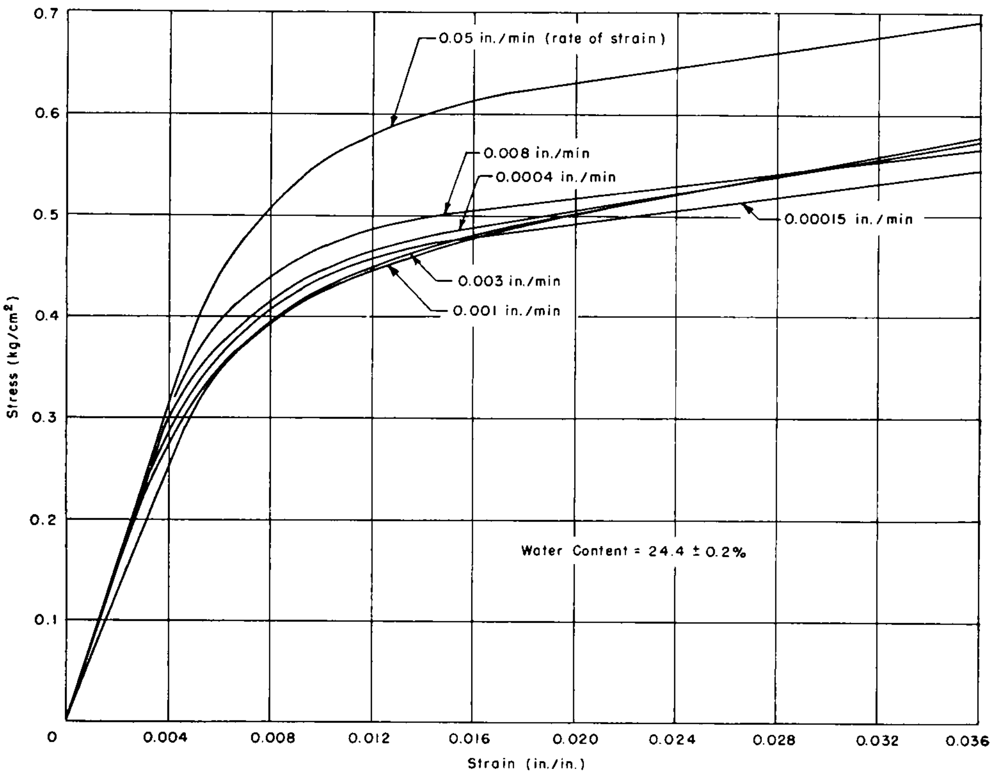

Figure 11. Initial portion of stress-strain curves corrected to zero; water content, 24.4 ± 0.2 percent.

water contents, the dry densities and degrees of saturation were not the same.

DISCUSSION OF RESULTS

The results indicate in simplest terms that as the rate of strain is decreased, both the maximum stress and yield point stress decrease at all water contents tested. Geuze and Tan (*2*) noted this effect in saturated remolded clays. Casagrande and Wilson (*3*) found the same result for several saturated undisturbed soils, but found that the strength of the compacted clays they tested increased as the rate of loading decreased. Seed and Chan (*4*) concluded that the observed increase in shear strength was due to thixotropic effects. They hypothesized that in a long-term test the soil strength was decreased by "creep deformation" and increased by thixotropic hardening, and whether the shear strength increased or decreased with long-time loading depended on the relative magnitudes of the two effects. The soil discussed herein would not be expected to exhibit much thixotropic hardening because it is an illitic soil, and all the specimens tested were at moisture contents much nearer the plastic limit than the liquid limit (*5*). This proved to be the case, as no significant difference was apparent between the strengths of specimens tested in a standard 10-min test a few hours after compaction and one month after compaction.

The initial tests were made without pore pressure measurements. Tests are now in progress in which pore pressures are measured to determine if curves of effective stress *vs* rate of strain give similar results.

Because the strength decreases with

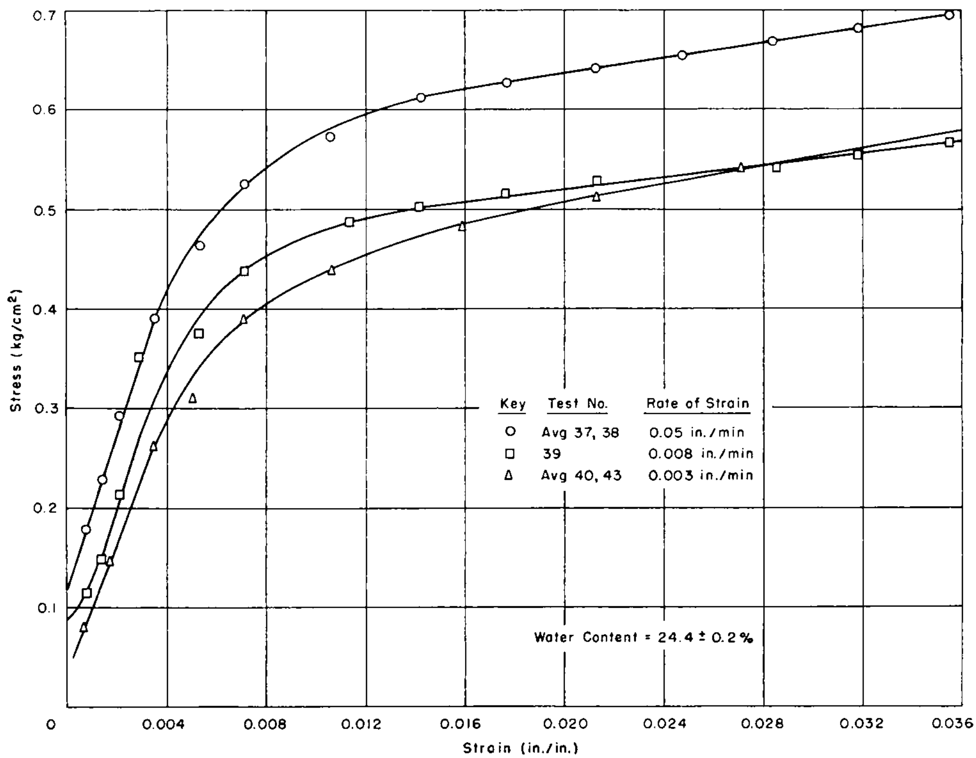

Figure 12. Typical initial portion of uncorrected stress-strain curves; water content, 24.4 ± 0.2 percent.

decreasing rate of strain, it was hypothesized that at constant stresses lower than the maximum, the compacted clay would yield with time and perhaps fail. De Beer (*6*) noted that the strength of a clay soil decreased when a constant load was applied for some time in the "Dutch cell" test. In contrast to his tests, a constant load less than the ultimate was applied to the soil and left there. The specimen was allowed to strain freely, but no drainage was permitted. However, because of the long test period involved, drying of many of the specimens was experienced, invalidating the test results. In two tests in which the loss of water was negligible, the results (Fig. 17) show that actual failure can occur at a constant stress considerably below the maximum. It is seen that at a water content of 24 percent constant loads of 40 percent and 60 percent of the maximum cause small continuous strain with time, but after a considerable time interval after load application the two specimens failed in a relatively short time. Although these tests are only preliminary, they indicate that at the water content tested (5 percent above optimum) a constant stress much less than the conventional unconfined strength can produce a shear failure, given sufficient time. A possible explanation for this phenomenon is that as strain occurs the structure dilates along the potential failure plane, causing an increase in moisture content in the failure area. Observations of the failure planes of the specimens tested indicated such a moist condition.

These results may possibly explain the failure of the embankment. If the soil as compacted in the embankment has a long-

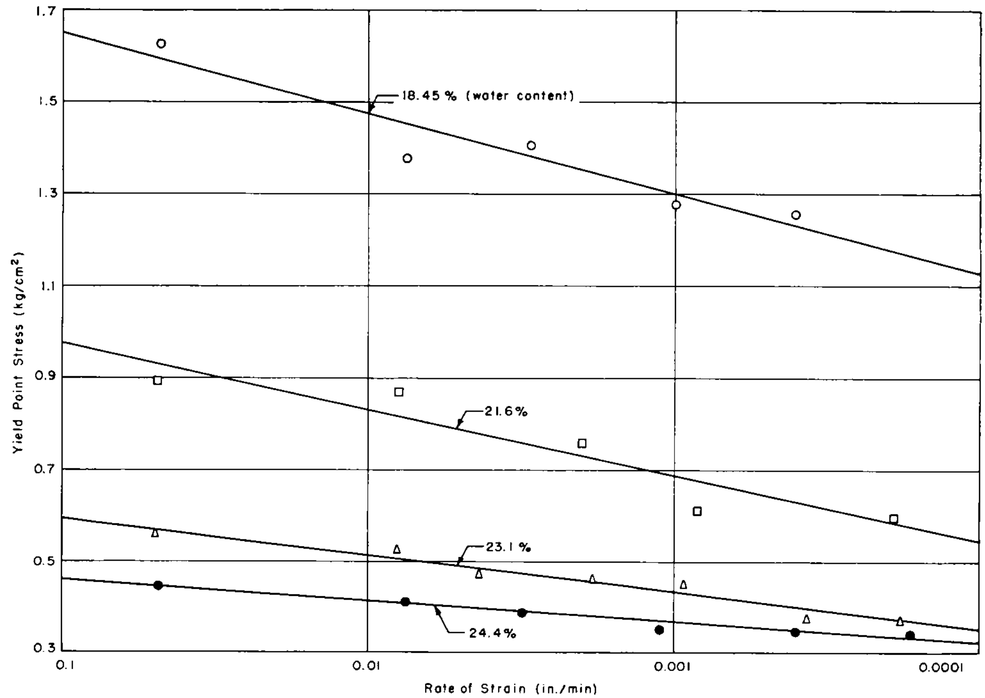

Figure 13. Relationship between yield point stress and rate of strain for various water contents.

time strength a little less than one-half the laboratory unconfined strength, and if shear failure is delayed, as in the laboratory constant-stress tests, an explanation of the failure appears possible.

CONCLUSIONS

1. At a given water content the compacted clay tested showed decreasing maximum strength and decreasing "yield point" strength with decreasing rate of strain. The strain at which failure occurred decreased markedly with decreasing rate of strain.
2. The modulus of deformation as indicated by the initial straightline portion of the stress-strain curve remains constant for varying rates of strain at a given water content.
3. The modulus of deformation *vs* water content plots a straight line at water contents greater than the shrinkage limit. (The soil, however, does not have the same dry density at all water contents.)
4. A few limited tests indicate that at a constant stress considerably less than the maximum strength obtained from a conventional unconfined test, failure can occur quite suddenly after a time interval.

ACKNOWLEDGMENT

The work reported herein was made possible in part by a research assistantship donated by the Caisson Corporation, Niles, Ill.

REFERENCES

1. WILSON, S. D., "Small Soil Compaction Apparatus Duplicates Field Results Closely." *Eng. News-Rec.*, 145:34-36 (Nov. 2, 1950).
2. GEUZE, E. C. W. A., AND TAN, T. K., "The Shearing Properties of

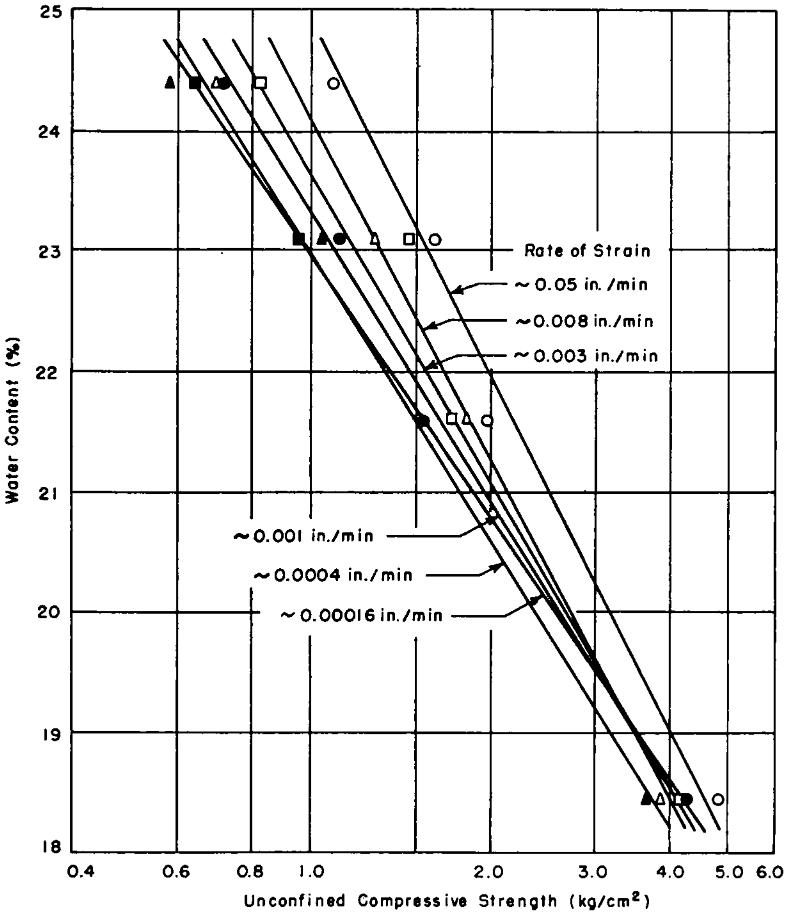

Figure 14. Relationship between unconfined compressive strength and water content for various rates of strain.

Soils." *Geotechnique,* 2:No. 2, 141-161 (Dec. 1950).

3. CASAGRANDE, A., AND WILSON, S. D., "Effect of Rate of Loading on Strength of Clays and Shales at Constant Water Content." *Geotechnique,* 2: No. 3 (June 1951).
4. SEED, H. B., AND CHAN, C. K., "Thixotropic Characteristics of Compacted Clays." A.S.C.E., *Jour. Soils Mech. and Found. Div., ASCE,* Vol. 83, Paper 1427 (Nov. 1957).
5. SKEMPTON, A. W., AND NORTHEY, R. D., "The Sensitivity of Clays." *Geotechnique,* 3: No. 1, 30-53 (Mar. 1952).
6. DE BEER, E. E., "The Cell Test." *Geotechnique,* 2: No. 2, 162-172 (Dec. 1950).

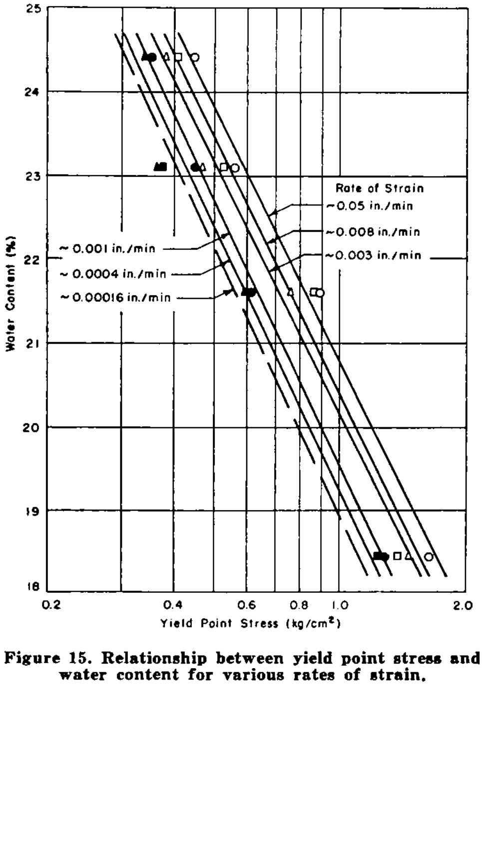

Figure 15. Relationship between yield point stress and water content for various rates of strain.

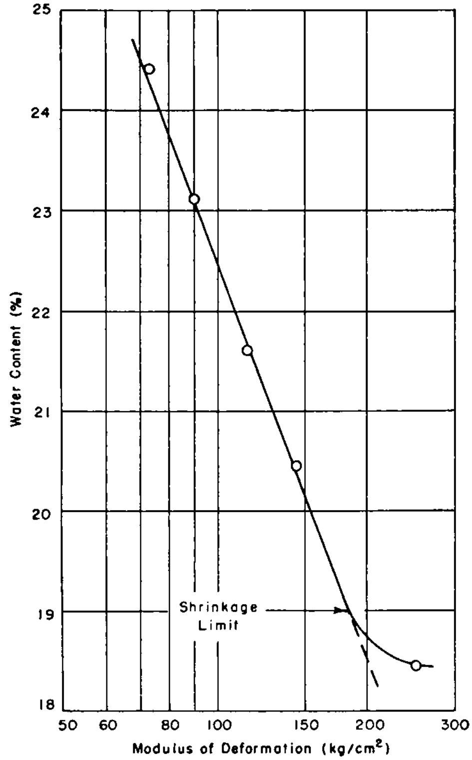

Figure 16. Relationship between modulus of deformation and water content.

DISCUSSION

J. K. MITCHELL and H. B. SEED, *respectively, Assistant Professor and Associate Professor of Civil Engineering, Institute of Transportation and Traffic Engineering, University of California, Berkeley.*—The excellent data presented by the authors constitute an important contribution to the increasing body of knowledge concerning the behavior of cohesive soils under varying durations and rates of loading. The test results shown in Figure 17 are an indication that the so-called "creep-strength" phenomenon, originally reported by Casagrande and Wilson (*3*) for saturated clays, occurs in partially saturated clays as well. A creep-strength loss occurs when a sample fails under a sustained stress of considerably smaller magnitude than the ultimate stress required to cause failure in normal compression tests.

Recent work in the writers' laboratory has indicated a very complex relationship between compressive strength and rate of loading in compacted clays. Figure 18 presents the compressive strength as a function of rate of loading for samples of silty clay prepared by kneading compaction to a density of 108.5 pcf and a water content of 18.5 percent. All samples were aged at constant water content for a period of 9 months prior to test in order to minimize any normal thixotropic strength increases during the test period. It may be seen that for increasing durations of test the strength at first decreases, then increases, and finally again decreases. It is conceivable that behavior

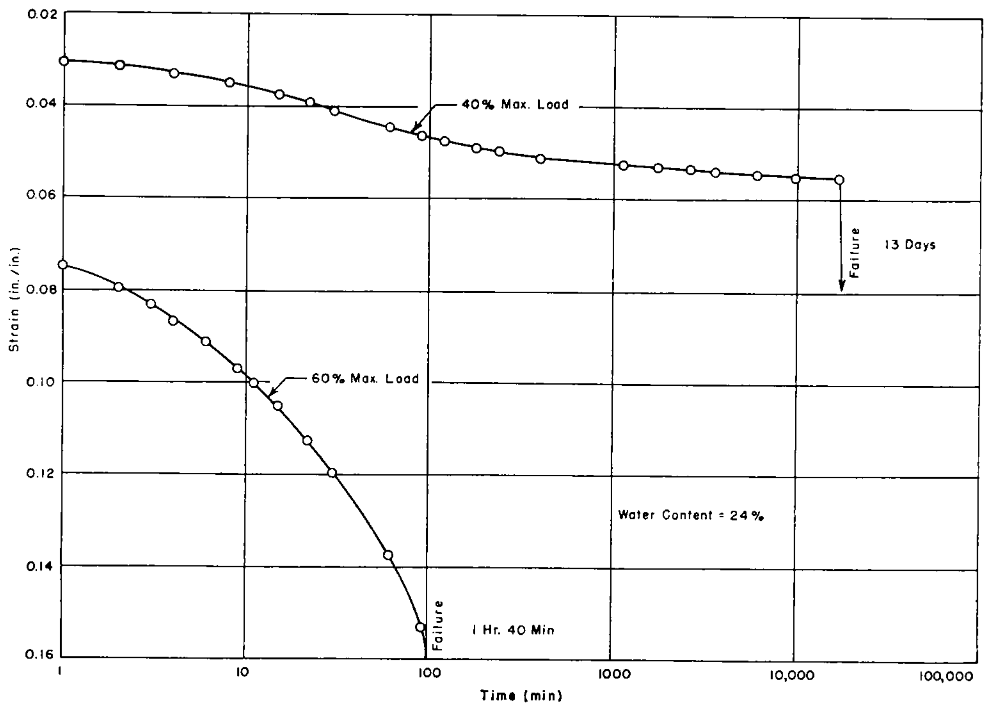

Figure 17. Relationship between strain and time for constant load.

of this type may result from the interplay of several phenomena, occurring to varying degrees under different rates of loading. For example, for increasing test durations the following factors may be important:

1. A tendency for strength to decrease due to increased time available for creep deformations.
2. A tendency for strength to decrease due to a manifestation of the creep strength phenomenon. Quite possibly the tendency for the soil structure to develop more brittleness under sustained loads is an important factor contributing to this phenomenon. Both the authors' data and results obtained in the writers' laboratory have indicated increased brittleness—i.e., failure at progressively lower strains—for samples failed under increasing durations of test.
3. A tendency for strength to increase due to normal thixotropic strength increases.
4. A tendency for strength to increase due to the regain of thixotropic strength lost at low strains.

The relative influence of these factors in any test series is, of course, dependent on such factors as soil composition, elapsed time between compaction and testing, type of test, and rate of loading. A more detailed consideration of effects such as those indicated in Figure 18 is presented in a forthcoming paper.*

In the light of these considerations it is suggested that the deviation of the points in Figure 7 from a straight line may not be entirely due to scatter, but may well reflect the combined effects of influences such as those suggested previously in this discussion.

The independence of the slope of the

* Seed, H. B., Mitchell, J. K., and Chan, C. K., "The Strength of Compacted Cohesive Soils." To be presented at Am. Soc. Civil Eng. Research Conference on Shear Strength of Cohesive soils (1960).

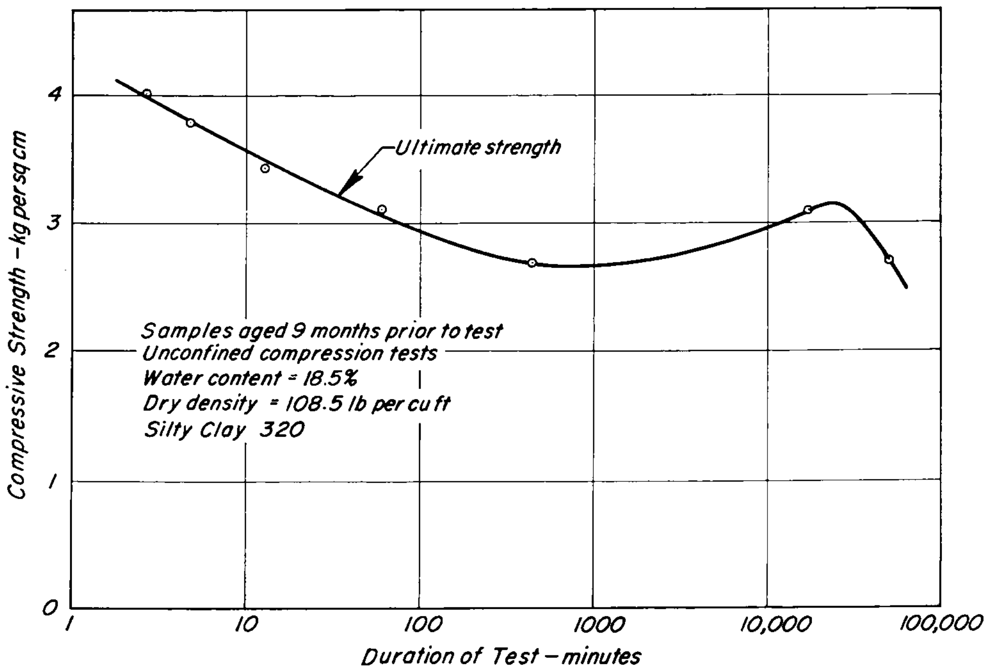

Figure 18. Effect of duration of test on compressive strength of compacted silty clay.

straightline portion of the stress-strain curves in Figures 8, 9, 10 and 11 with respect to strain rate is most interesting. This result is suggestive of a structure that behaves almost completely elastically at very low strains.

Tests with measured pore pressures, now being conducted by the authors, should not only be of great value in explaining strength behavior under varying rates of loading but may also provide a better insight into the fundamental factors influencing shear strength in general.

J. O. Osterberg and William H. Perloff, Jr., *Closure*.—The interesting comments by Professors Mitchell and Seed certainly shed more light on the relationship between shear strength and rate of loading for compacted clay soils.

Although the authors agree that all four factors listed by the discussers affect the general case, factors 3 and 4 probably played little if any part in the soil reported, because this soil exhibited no thixotropic strength increase with time. Thus, Figure 7 appears to indicate only the effect of "creep deformations" on strength. Furthermore the deviations from the straight line appear to be actual scatter, inasmuch as these deviations become very small at the higher water contents, where thixotropy would presumably have more, not less, effect. It is interesting to note that the four shortest tests in Figure 18 also lie close to a straight line. It seems likely that these four tests would also show the effect of creep deformation only, as the specimens were all aged to eliminate normal thixotropic strength increases during the test period, and these tests were probably too rapid to permit thixotropic regain of strength lost at low strains. Again, deviations from the straight line are not large.

It is gratifying that Professors Mitchell and Seed corroborate the authors' observations of increasing brittleness with decreasing rate of loading, for compacted clay. It is hoped that work now in progress will shed more light on this phenomenon.