LABORATORY COMPARISON TESTS USING COMPACTED FINE-GRAINED SOILS

ESSAIS COMPARATIF DE LABORATOIRE UTILISANT DES SOLS A GRAIN FIN COMPACTES

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SYNOPSIS Comparative tests on typical fine-grained soils from selected borrow areas revealed that significant variations in results can occur among different Corps of Engineers laboratories using virtually identical soils. These variations in results of classification, compaction, triaxial compression, and direct shear tests can be attributed to differences in testing techniques and equipment and to minor differences in the soil samples. Thus, comparative testing among other laboratories to determine the degree of variation is suggested.

INTRODUCTION

The U. S. Army Corps of Engineers maintains nine major soil testing laboratories in the United States. The primary purpose of these laboratories is to perform tests for the determination of values used in the design of projects built under the supervision of the Corps. In order to obtain information on the variation in test results when the laboratories performed the same tests on nearly identical soils, three types of soil were sent to each of the laboratories.

SOILS TESTED

The materials selected for the program were taken from borrow areas near Vicksburg, Mississippi, and represent typical fine-grained soils of the Lower Mississippi Valley. The three soils were a brown fat clay, CH (buckshot), a brown lean clay, CL, and a tan silt, ML (loess). The CH soil was passed through a 1/4-inch screen, aerated to a nearly dry state with a pulvimixer, mixed thoroughly with hand tools, and spread into a 15-cu-yd stockpile. Each of 333 containers was filled with 80 pounds of CH soil that was taken, a few pounds at the time, from random locations in the stockpile. The entire procedure was repeated for the CL and ML soils. A sealed container of each of the three soil types was shipped to the nine Division laboratories and the following tests were performed:

a. Atterberg limits, specific gravity, grain-size analysis, and standard compaction.

b. Unconsolidated-undrained triaxial compression (Q tests).

c. Consolidated-drained direct shear (S tests).

The testing procedures were to be in accordance with those outlined in the Corps engineering manual "Laboratory Soils Testing." The confining pressures for the Q tests and the normal pressures for the S tests were to be 0.5, 1.5, 3.0, and 5.0 tons/sq ft. The soils for the Q and S tests were to be molded, using kneading compaction, to the following conditions:

Soil Type	Dry Unit Weight, pounds/cu ft	Molding Water Content, percent
Silt (ML)	100.0	19.0
Lean Clay (CL)	103.5	18.5
Fat Clay (CH)	93.0	24.5

The results of all tests and a detailed description of the techniques and equipment used were sent to the Office of the Chief of Engineers and a representative of that office visited each laboratory at the completion of the test program to discuss the results and inspect the equipment. All Q tests were performed on specimens about 1.4 inches in diameter and 3.0 inches high and all S tests were made on specimens that were 3.0 inches by 3.0 inches and 0.5 inches thick.

CLASSIFICATION TEST RESULTS

As shown in fig. 1, there is a comparatively wide spread in grain-size curves for all three soil types. The variations result from accumulative errors including those in operator technique, methods of slaking and dispersing the specimen before placing in the cylinder, types of dispersing agents used, calibration of hydrometers, and inherent differences in the soil. In spite of elaborate mixing and sampling procedures, the soils sent to each laboratory were not identical. There are insufficient data to determine, statistically, the extent to which nonuniformities in the soil contributed to the variations in test results, since that was not the purpose of this test program.

The differences in the specific gravity values, which range from 2.65 to 2.73 for the ML soil, from 2.60 to 2.76 for the CL soil, and from 2.61 to 2.74 for the CH soil, are attributed mainly to different techniques used in deairing the soil-water mixtures. Other contributing factors include errors in weighing and calibration of the pycnometer.

As shown in fig. 2, the liquid limit values range from 49 to 60 for the CH soil, from 30 to 36 for the CL soil, and from 27 to 29 for the ML soil. The greatest errors were introduced in the test by incorrect height of fall of the cup and improper forming of the groove

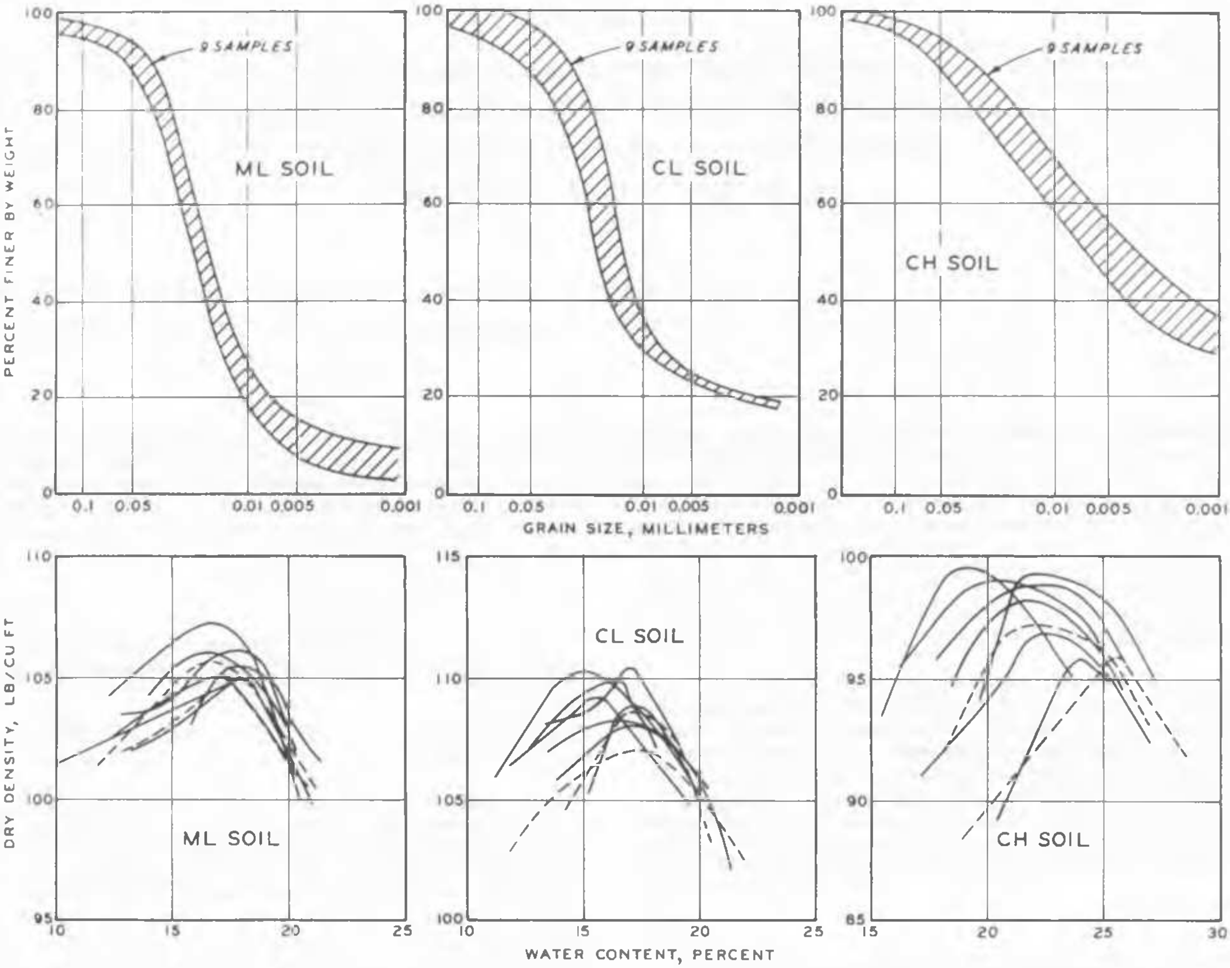

Fig. 1 Grain-size distribution and compaction curves

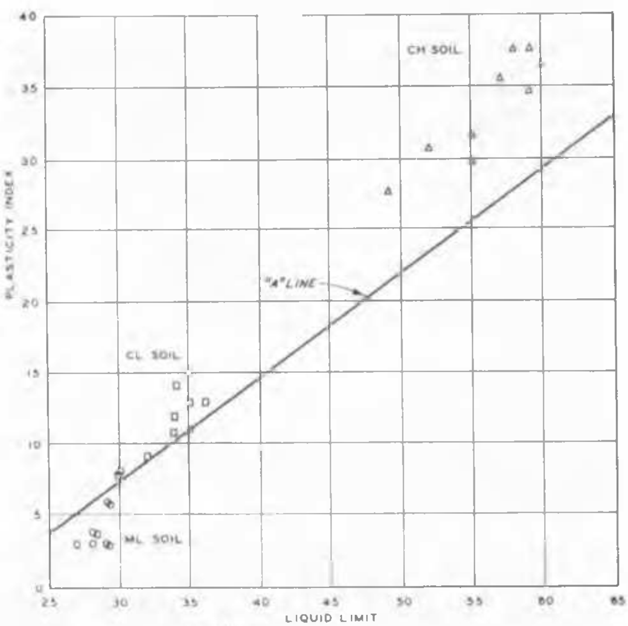

Fig. 2 Plasticity chart

in the soil pat. Other factors that contributed to the variation in liquid limit values among the nine laboratories are insufficient mixing and curing of specimen, loss of colloidal material during processing, inaccurate blow count, improper length of groove closure, worn cup or grooving tool, incorrect dynamic resiliency of the base of the liquid limit device, and sample drying that could inhibit the flow of the two halves of the soil pat. The range of plastic limit values is from 21 to 25 for the CH soil, from 20 to 23 for the CL soil, and from 23 to 26 for the ML soil. The main causes for these variations were incorrect final thread diameter and length caused by rolling soil with the fingers rather than with the palm. Rolling the threads with the fingers causes premature separation due to unconscious spreading and to concentration of pressure.

COMPACTION TEST RESULTS

Each laboratory performed the standard effort compaction test, using a 5.5-pound hammer with a 12-inch drop, using three soil layers in a 1/30-cu-ft mold. In fig. 1 are plotted the results for each soil type. It was from the results of these tests that molding conditions were selected for the triaxial compression and direct shear tests. The predominant factors contributing to the rather wide variations in compaction results were use of different types or models of compaction hammers (most of which are mechanical hammers), unevenness in the thickness of the three soil layers, excess "strike-off" of compacted material above the top of the compaction mold, and differences in the pattern of hammer blows.

UNCONSOLIDATED-UNDRAINED (Q) TRIAXIAL TEST RESULTS

Plots of the shear strength envelopes from the nine laboratories are presented in fig. 3. The range in shear strength values is as follows:

Soil Type	Angle of Internal Friction ϕ_Q, degrees	Cohesion, C_Q, tons/sq ft
Silt (ML)	27.6-32.2	0-0.40
Lean Clay (CL)	16.0-24.0	0.40-0.80
Fat Clay (CH)	1.5-5.8	0.60-1.00

The envelopes were drawn tangent to the four Mohr's circles which were based on the maximum deviator stress. For those tests in which the deviator stress continued to increase with increasing strain, the

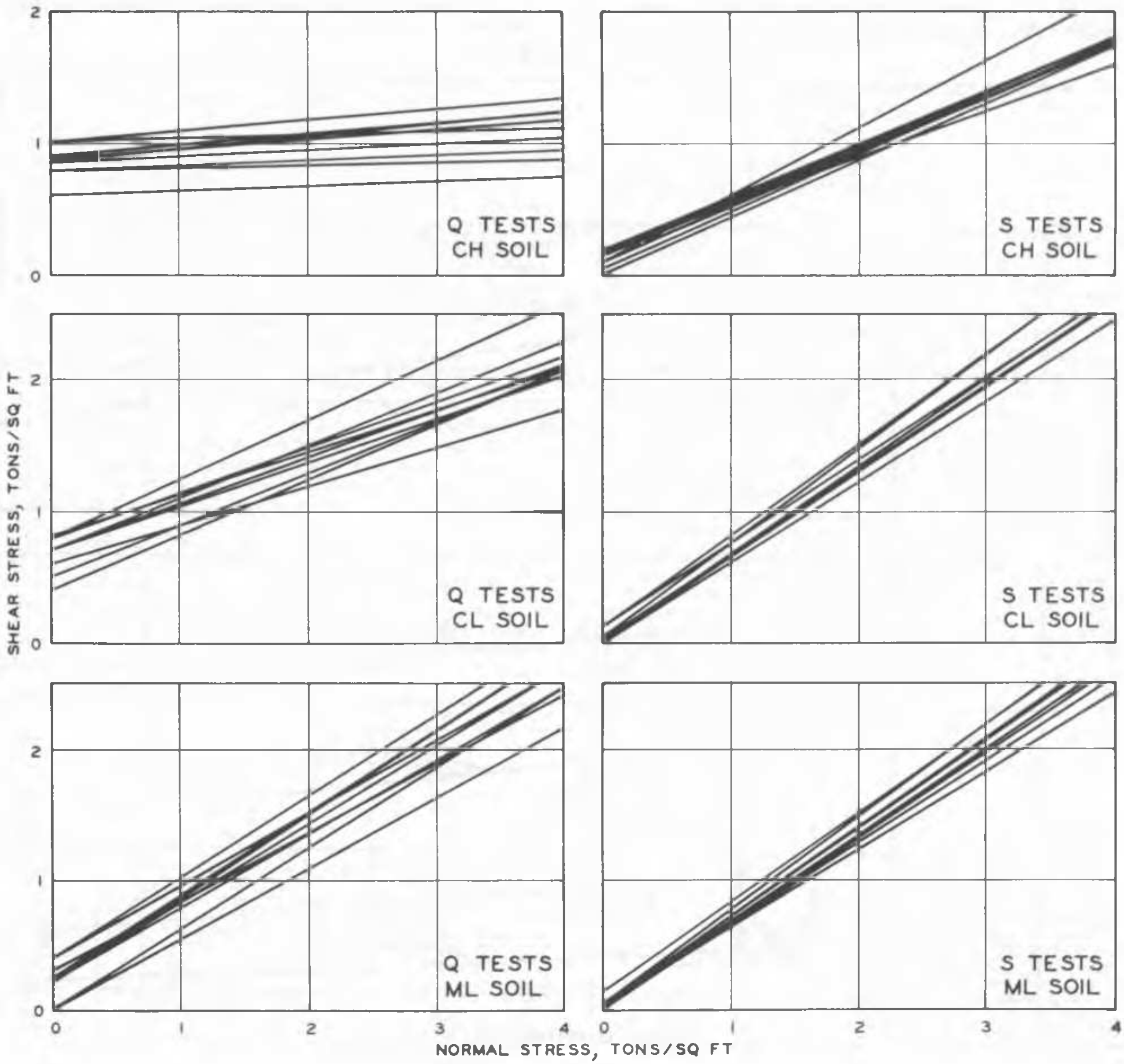

Fig. 3 Shear envelopes for Q and S tests

maximum deviator stress was taken to be that at 15 percent strain, since shear strengths beyond this value are considered to have little meaning because of sample deformation. The variations in the results of these Q tests can be attributed mainly to the three factors discussed in the following paragraphs:

a. Effect of rate of strain. Although the rate of strain recommended for these soils was 1 percent per minute, the laboratories actually used rates of strain ranging from 0.29 to 1.09 percent per minute. This range of strain rates permitted a plot of time to maximum deviator stress versus maximum shear strength. Fig. 4 indicates that for these molding conditions the Q strength of the CH and CL soils are time dependent and show a decrease in strength with an increase in time to maximum deviator stress. A similar pattern could not be detected from a plot of the test results on the ML soil, because of the ability of this material to achieve near equilization of pore water pressure during rapid shear.

b. Effect of as-molded water content. Specimens of the CH soil were compacted at water contents ranging from 23.6 to 27.1 percent. The molding water contents for the CL soil varied from 19.2 to 21.8 percent rather than being compacted to 18.5 percent water content as requested. Fig. 5 shows that because of these differences, for the CL soil there was a decrease in shear strength with increase in initial water content, as would be expected in clays. Only an inferred trend of strength decrease with molding water content increase can be seen in the plot of the results of the tests on the CH soil (fig. 6); a similar plot of data on the ML soil showed a scatter without pattern.

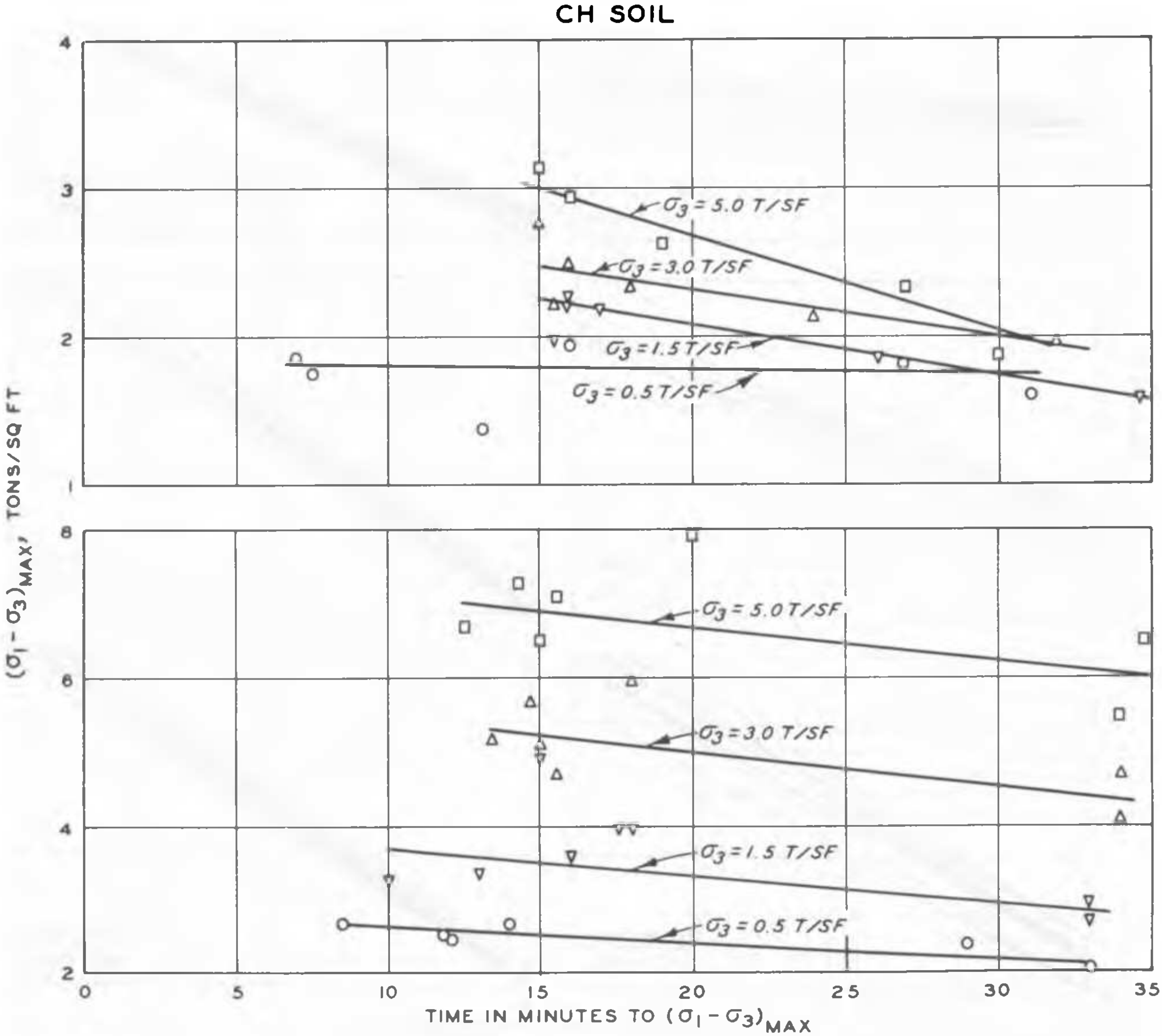

Fig. 4 Effect of strain rate on Q shear strength

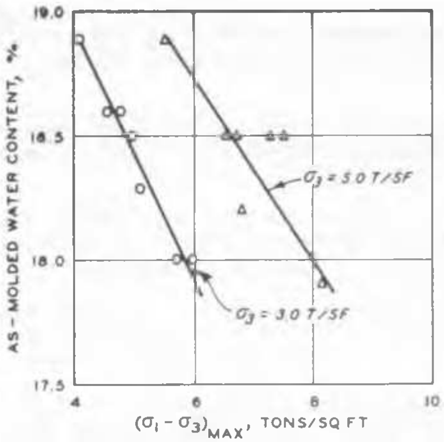

Fig. 5 Effect of molding water content on Q shear strength, CL soil

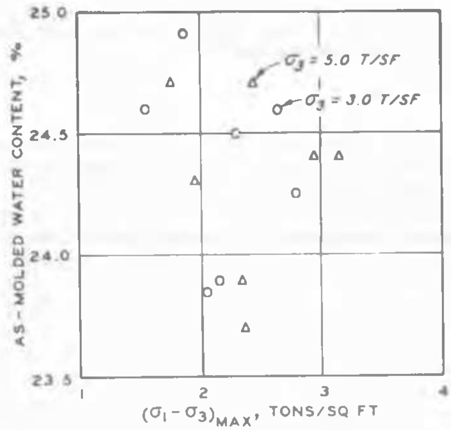

Fig. 6 Effect of molding water content on Q shear strength, CH soil

c. <u>Effect of type of loading platen.</u> At the time of this testing program, the allowable degree of freedom of the upper platen used in the triaxial test had not been standardized in the Corps. A comparison of results using two types of loading platens is shown in fig. 7. The molding water content, dry unit weight, and rate of strain for these specimens were nearly identical. When a platen with a small allowable tilt is used in the triaxial test, the mode of failure, the maximum deviator stress, the shape of the stress-strain curve, and the magnitude of the strain at failure are entirely different from those of tests in which a free-tilting platen is used. A ball-and-socket type of loading cap tends to give less end restraint but may contribute to localized failures that usually take place within the upper third of the test specimen. More research is necessary to investigate the significance of the type of platen used.

d. <u>General.</u> Other factors that contributed to the variance in test results include the molding method used to form the specimens, effect of piston friction, membrane restraint, membrane leakage, curing time of specimen before shear, temperature of the chamber water, specimen disturbance, area correction at high strains, and proving ring calibration.

CONSOLIDATED-DRAINED (S) DIRECT SHEAR TEST RESULTS

Fig. 3 shows the direct shear failure envelopes for each soil type. The relatively small variations in results from the nine laboratories reconfirm the fact that reproducibility of results in S tests is more certain than in Q tests. The factor that contributed most to the differences in results in these direct shear tests was the variation among the laboratories of the molding water contents. These initial water contents affect the consolidated unit weight, the soil structure, and, indirectly, set the criterion for rates of strain. For controlled-strain direct shear tests, strain rates are established as a function of the time-rate of consolidation before shear, which is influenced by the initial molding water content. Another variable causing differences in test results is the partially saturated state of the specimens during shear under low normal pressures. In direct shear tests of compacted soils, complete saturation is induced by decreasing the volume of the voids, since back-pressure is not employed. The soils tested had initial degrees of saturation as low as 73 percent and some specimens with normal pressures of 0.5 and 1.5 tons/sq ft were sheared in a partially saturated condition. Further,

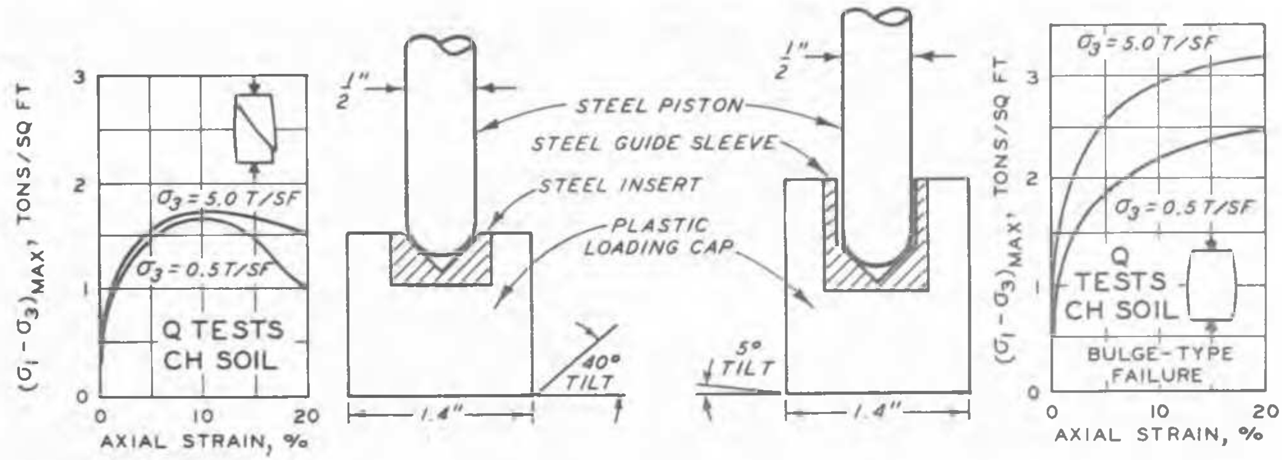

Fig. 7 Effect of type of loading cap in triaxial tests

some differences in results are due to nonuniform densities, uneven stress distributions, and area variations induced during shear.

CONCLUSIONS

There are insufficient data from this comparative test program to make statistical studies of the deviations. Further, it is impossible to use these results to differentiate between variations due to operator techniques, equipment, or minor differences in the soil structure. However, some of the main causes for the errors have been determined and a qualitative analysis of results is possible. Even the most experienced laboratory can, by the inadvertent introduction of seemingly minor errors in testing techniques and equipment, produce end results significantly different from those produced by other laboratories. Wherever possible, routine testing procedures for deriving design values should be standardized and every phase of the methods and equipment used should be examined regularly to discover reasons for deviations in results among different laboratories. The engineer must take into account the inevitable errors inherent in all soils testing and comparative test programs should be encouraged.

REFERENCES

Bishop, A. W. and Henkel, D. J., (1962), "The Measurement of Soil Properties in the Triaxial Test."

Casagrande, A., et al., (1963), "Investigation of Stress-Deformation and Strength Characteristics of Compacted Clays," Harvard Soil Mechanics Series No. 70.

Department of the Army, Office of the Chief of Engineers, Engineer Manual 1110-2-1906, "Laboratory Soils Testing," (1965).

Duncan, J. M. and Seed, H. B., (1965), "Errors in Strength Tests and Recommended Corrections," Report No. 3-133, University of California.

Duncan, J. M., et al., (1966), "The Significance of Cap and Base Restraint in Strength Tests on Soils," Report No. 3-159, University of California.

Johnston, M. M. and Strohm, W. E., (1968), "Results of Second Division Laboratory Testing Program on Standard Soil Samples," Miscellaneous Paper No. 3-978, U. S. Army Engineer Waterways Experiment Station.

Lambe, T. W., (1951), "Soil Testing for Engineers."

Strohm, W. E., (1966), "Preliminary Analysis of Results of Division Laboratory Tests on Standard Soil Samples," Miscellaneous Paper No. 3-813, U. S. Army Engineer Waterways Experiment Station.

Taylor, D. W., (1948), "Fundamentals of Soil Mechanics."