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ed by 15,000 ft lb per cu ft, will have an indicated saturated penetration resistance of 300 lb per sq. in., and Fig. 3 shows a consolidation under a 50 lb per sq in. loading of 3.9% for soils having an indicated saturated penetration resistance of 300 lb per sq in.; accordingly, 3.9% consolidation has been plotted on Fig. 6 at 15,000 ft lb per cu ft compaction effort on the sandy soil. The anticipated soil consolidation from the use of "90, 95, and 100% compaction" shown on Fig. 7 has been shown on Fig. 6 for the 12,375 and 56,250 ft lb per cu ft compaction methods.

CONCLUSION

The data from which the foregoing discussion was prepared have been accumulating over

a considerable period of years and have been assembled into the various figures in an effort to learn more of the relationship between compactive effort, soil density, and soil consolidation, and to find reasons why similar results were not always secured from soil compaction where apparently the same methods were used.

REFERENCES

- 1) R.R. Proctor, Laboratory Soil Compaction Methods, Penetration Resistance Measurements, and the Indicated Saturated Penetration Resistance. Paper to this Conference.
- 2) See Paper IXb 14 for further discussion of the relationship between field and laboratory compaction.

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IX b 13

A COMPARISON OF THE PHYSICAL PROPERTIES OF AN ALLUVIAL SILT COMPACTED BY FIELD AND LABORATORY METHODS

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SYNOPSIS

A comparison of the physical properties of a soil compacted by field and laboratory methods is made in this paper. From an investigation of shear, consolidation, and permeability characteristics, it is shown that there is little difference in the physical properties of a field and laboratory compacted alluvial silt.

INTRODUCTION

The present trend in earthwork construction makes the investigation of this problem very desirable, particularly, if the design of projects is to be based on the results of laboratory compaction tests.

The characteristics of the particular soil examined were determined by the unconfined compression, triaxial shear, consolidation, and permeability tests which were performed on soil compacted by field and laboratory methods. All tests were performed at approximately the same moisture content and density. Testing procedures described by Casagrande in "Notes on Soil Testing for Engineering Purposes" were followed. 1)

SELECTION AND CLASSIFICATION OF THE SOIL

A highway embankment constructed of an alluvial silt was chosen as the source of material for the research. Field compaction was accomplished with a double-unit sheepsfoot roller developing a working pressure of three hundred pounds per square inch on the area of each foot. No moisture control was exercised during construction. Final construction was completed on February 8, 1947.

Undisturbed samples were removed from the embankment by the method described by Bertram, in "Soil Tests for Military Construction", applying to "chunk samples". 2) Average dry density and moisture content values for the field compacted soil were 102.0 pounds per cubic foot and 16.2 percent, respectively. Average field compaction of the soil was 93 percent of the Modified AASHTO method and 96 percent of the

Standard Proctor method. The compaction characteristics of this material are shown by the curves in Figure 1.

Soil constants, which were determined by standard testing procedures 3), and the grain-size distribution curve are shown in Figure 2. The soil was classified as an A-4 material using the U. S. Public Roads Classification System 4) or an ML material by the Army Classification System. 5)

SHEAR CHARACTERISTICS IN AN UNCONFINED STATE

A comparison of the shear characteristics for field and laboratory compacted soil in a non-confined state was made. Unconfined compression tests were performed with a Universal Testing Machine so controlled that the average rate of strain for the tests was fifteen-thousandths of an inch per minute and the average time for failure was six minutes. Loads were measured to the nearest 0.25 pound.

A comparison of the unconfined compressive strength for field and laboratory compacted soil is shown in Table 1. Although there is considerable variation in the modulus of elasticity and unit strain at failure for field and laboratory compacted soil, the average values for these two factors are in relatively close agreement.

Despite the fact that slightly greater densities and lower void ratios were obtained for the laboratory compacted soil, the maximum ultimate strength of the field compacted soil was approximately ten percent higher than for the laboratory compacted soil. The greater ultimate strength was attributed to a phenomenon

TABLE 1
RESULTS OF UNCONFINED COMPRESSION TESTS

Specimen No.	Moisture Content percent	Modulus of Elasticity lb. per sq. in.	Void Ratio e	Compressive Strength lb. per sq. in.	Strain at Failure percent
1 x)	17.2	605	657	11.95	2.35
2 x)	16.2	640	673	11.95	2.20
3 x)	16.7	585	643	11.56	3.05
4 x)	16.7	625	636	11.80	3.00
Average	16.7	614	652	11.81	2.65
11	15.9	800	600	11.95	2.80
12	15.0	860	624	10.92	2.30
13	15.2	800	608	12.35	2.65
14	17.4	550	605	10.40	2.80
15	17.0	430	593	9.88	2.90
16	17.0	390	610	9.72	3.50
17	16.2	600	609	9.25	2.65
18	15.4	570	538	11.80	2.65
19	17.2	490	592	10.75	2.85
Average	16.2	610	598	10.78	2.79

x) Field Compacted Soil

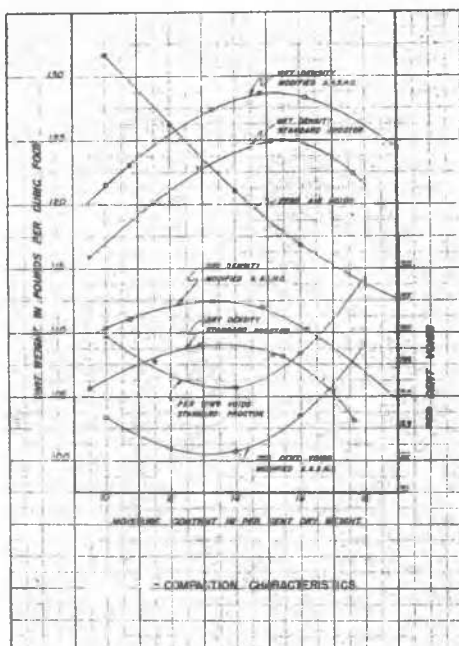


FIG. 1

termed "dry curing" by McDowell. 6) Dry curing was attributed to a change in moisture film characteristics of the individual grains rather than the effects of surface tension and changes in arrangement of soil particles. 7)

A reduction in moisture content resulted in an increased bond (apparent cohesion) between the soil particles caused by the capillary action of the moisture film surrounding the individual grains. Resistance to shear was obviously increased by this cohesion. Stress versus strain curves in Figure 3 illustrate the effect of this phenomenon.

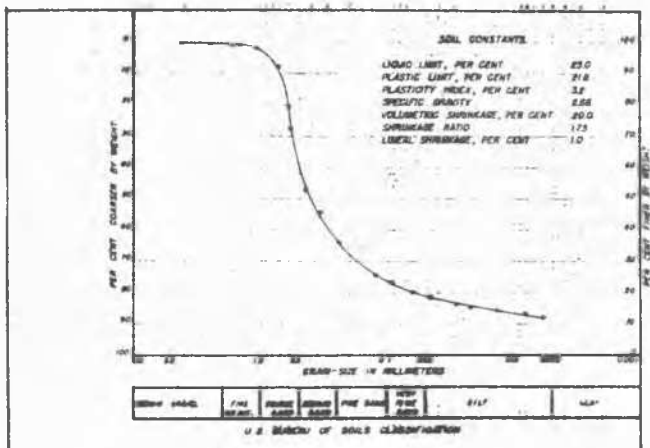
McDowell showed that the effect of the phenomenon was not entirely destroyed as the soil regained moisture from seepage or capillarity. 6)

SHEAR CHARACTERISTICS AS SHOWN BY TRIAXIAL TESTING

A comparison of the major principal stresses and the angles of internal friction for field and laboratory compacted soil was made from triaxial shear test data. Tests were performed at lateral pressures of 10, 20, and 30 psi.

The average rate of strain was two-hundredths of an inch per minute with failure occurring in approximately ten minutes.

Stress versus strain characteristics for triaxial shear tests are shown in Figure 4. These curves indicate that the ultimate strength



GRAIN-SIZE DISTRIBUTION CURVE

FIG. 2

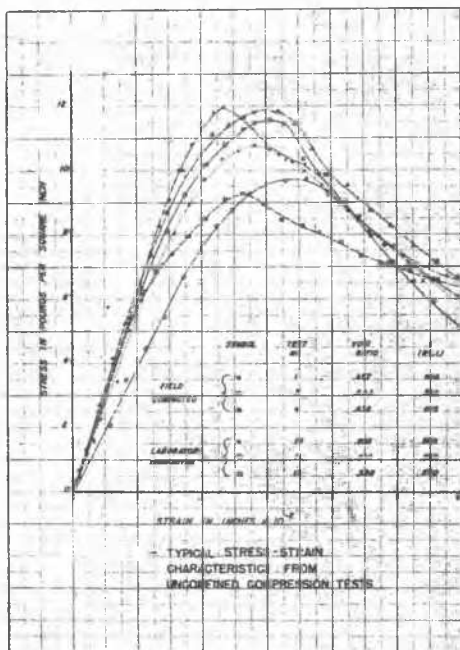


FIG. 3

was influenced by changes in void ratio. There was no appreciable difference in the ultimate strength of field and laboratory compacted soil as determined by triaxial testing. The so-called "dry cured strength", present for unconfined compression tests of field compacted soil, did not appear to affect the shear strengths. Saturation of the samples was undoubtedly responsible for eliminating any apparent cohesion developed by capillary action. Hence the ultimate strength values obtained by triaxial testing were dependent on intergranular stresses and true cohesion. Unconfined compression and triaxial shear tests data shows that ultimate strength values were influenced much more by apparent cohesion than by true cohesion.

The state of stress in each sample at time of failure was determined graphically by applying the principles of Mohr's Circle of Stress. This showed that the angle of internal friction varied inversely both as the lateral pressure and void ratio.

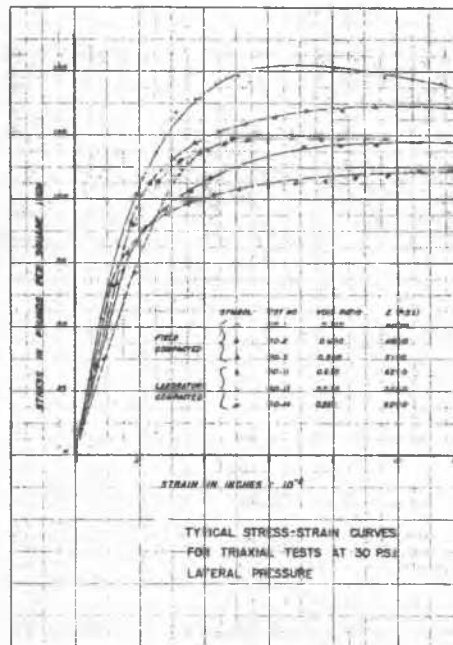
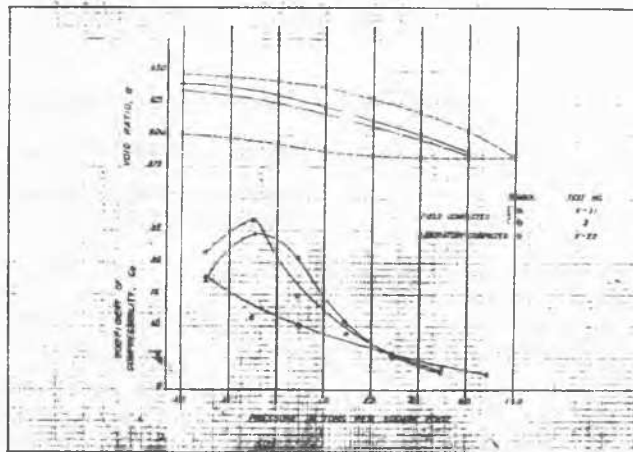


FIG. 4



Typical consolidation characteristics of field and laboratory compacted soil

FIG. 5

CONSOLIDATION CHARACTERISTICS

A comparison of the consolidation characteristics for field and laboratory compacted soil was made by determining the volume change and rate of volume change a laterally confined soil sample would undergo when subjected to an axial load.

Typical void ratio versus pressure and coefficient of compressibility curves are shown in Figure 5. The coefficients of compressibility, defined as the slope of the void ratio versus pressure curve, are tabulated in Table 2. The coefficients used were average values obtained by assuming a straight line variation between successive load increments. The data for consolidation tests performed on samples with the applied load perpendicular to the normal testing plane were also included in this table. Although the primary purpose of the horizontal consolidation tests was for permeability, a close similarity of test results was noted and

TABLE 2
COEFFICIENTS OF COMPRESSIBILITY AS DETERMINED
BY CONSOLIDATION TESTS xa)

Specimen No.	Load in tons per square foot					
	.125-.25	.25-.5	.5-1	1-2	2-4	4-8
V-3 xa)	17.6	11.2	10.0	8.8	5.7	3.8
V-4 xb)	15.2	16.2	14.0	10.4	6.8	4.9
V-11 xb)	16.8	24.4	21.0	10.5	5.0	3.1
V-12 xb)	22.4	28.0	32.4	15.3	9.8	4.2
V-13 xb)	20.8	29.6	15.0	8.5	6.1	4.7
H-101 xb)	15.6	44.0	18.4	7.2	9.5	3.1
H-102 xb)	26.4	18.0	12.2	7.6	3.8	2.1
V-5	25.6	24.8	19.2	8.6	6.4	3.7
V-21	20.8	24.0	17.0	4.2	2.6	1.9
V-22	21.6	26.8	14.8	10.3	5.1	2.9
H-111	21.6	8.1	7.8	10.2	8.6	3.1
H-112	13.2	16.8	13.8	11.3	8.0	4.7

xa) Values in table are times 10^{-3}

xb) Field Compacted Soil

H----Indicates tests were performed with direction of applied load perpendicular to normal testing direction

for this reason all consolidation test data were grouped in Table 2.

Test data failed to show an appreciable difference between the consolidation characteristics of field and laboratory compacted soil. Almost identical amounts of consolidation occurred for each increment of load for all tests. This was also true for the rebound portions of void ratio versus pressure curves. It is believed that saturation of the test specimens was instrumental in reducing the effectiveness of "dry cured strength" incurred through capillary action. Surface tension, which would cause a reduction in test sample thickness as drying occurred, was also relieved by saturation of the samples.

PERMEABILITY CHARACTERISTICS

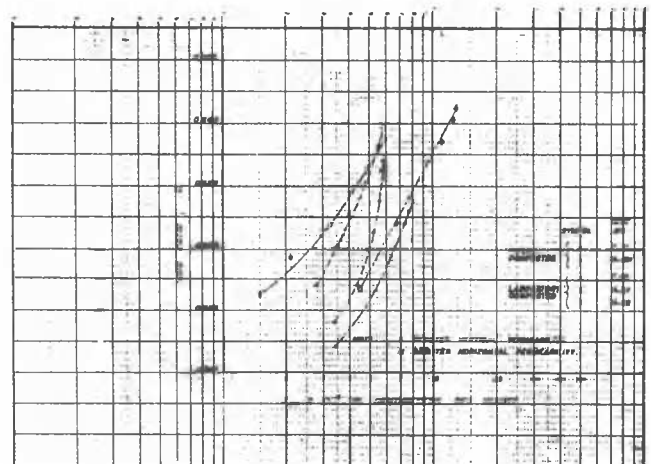
A comparison of the susceptibility of field and laboratory compacted soil to the passage of de-aerated water was made.

Coefficients of permeability were determined for flow in both a vertical and horizontal direction, and are denoted by k_v and k_h , respectively. The coefficients of permeability versus void ratio relationships, which approach a straight line, are shown in Figure 6.

A comparison of the results for permeability tests performed on field and laboratory compacted soil failed to reveal any consistent differences. This was true for permeability in both a vertical and horizontal direction.

CONCLUSIONS

The following conclusions are based on



Permeability characteristics

FIG. 6

tests performed on a typical inorganic, alluvial silt (ML material), and are not necessarily true for other types of soils.

A) There was no appreciable difference in the physical properties of field and laboratory compacted soil insofar as could be detected from consolidation, permeability, and triaxial shear characteristics as determined by this research.

B) There was a small difference in the ultimate

strength values of field and laboratory compacted soil as determined by the unconfined compression tests. The greater value of ultimate strength for field compacted soil was attributed to "dry curing".

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- 6) McDowell, Chester, Progress Report on Development and Use of Strength Tests for Subgrade Soils and Flexible Base Materials, Proceedings, Highway Research Board, 26: 484-506, 1946.
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THE RELATIONSHIP BETWEEN THE FOOT POUNDS PER CUBIC FOOT OF COMPACTIVE EFFORT EXPENDED IN THE LABORATORY COMPACTION OF SOILS AND THE REQUIRED COMPACTIVE EFFORTS TO SECURE SIMILAR RESULTS WITH SHEEPSFOOT ROLLERS

IX b 14

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SYNOPSIS

There are presented herein the results of tests to determine the laboratory compactive efforts (ft lb per cu ft) required to duplicate the actual construction results (soil dry weights) secured during the construction of six major dams in Southern California (involving about 12,000,000 cu yd of compacted earthfill together with a comparison of these compactive efforts with the construction compactive efforts as calculated by multiplying the known or estimated (by use of manufacturer's tables) drawbar pulls by the number of roller trips over each foot of depth of compacted soil and dividing the product by the roller widths.

FIELD AND LABORATORY TEST METHODS

Field test methods.- The soil samples were secured in one case from preserved samples of a test fill (Dam SG), in another case (Dam LV), from preserved samples from a year's series of compacted soil density tests, and in the four other cases by auger hole samples of the finished dams. The compactive efforts by the rollers (ft lb per cu ft) were calculated by multiplying the drawbar pull by the number of roller trips per foot of depth of compacted fill and dividing by the width of the roller. The roller drawbar pull was measured in the case of Dam SG and B1. The other drawbar pulls were taken from the manufacturer's tables for the speeds at which it was known that the tractors used could pull the rollers. These values were, in general, the next higher speed than the one used because of the necessity for the rollers to turn; that is, if a tractor could pull a roller in 4th gear at 2.7 m.p.h. (13,344 lb drawbar pull) it would usually be necessary to use 3rd gear at 2.3 m.p.h. (15,580 lb drawbar pull capacity) in order to negotiate turns, in which case 13,344 lb would be used in the compactive effort calculations.

Laboratory test methods.- The soil samples were compacted in the laboratory in 1/20 cu ft containers by the use of 5-3/4 lb tampers with 2 inch diameter striking faces dropped the correct distances to cover applicable values in the range of 10,000 to 50,000 ft lb per cu ft. 1), 2), 3).

DISCUSSION OF TEST RESULTS

Description of test results shown in Fig. 1.- Fig. 1 shows typical results for a test fill wherein both field and laboratory soil dry weights are plotted in the same figure. The "soil" used here was a soft, metamorphosed dioritic or granitic gneiss that had been selected originally as the quarry site for a rock fill dam but which, while appearing to be sound rock by visual inspection, was disclosed under the petrographic microscope to have been so badly crushed by pressure that small, healed cracks occurred every millimeter or so in the rock mass. The material had, of course, been blasted before removal from the "quarry" site and the larger than 3 inch portions removed by a grizzly. No difficulty was experienced in compacting this "soil" (2% passing the 200 mesh sieve), except to make the necessary tests to determine the required roller weight, number of roller trips, and the area of roller teeth; 24,200 lb gross weight was found necessary for each 5 ft long roller drum with a consequent 6000 lb drawbar pull (25% of the roller weight) for each drum, 12 sq in roller teeth were required instead of the usual 6 to 7 sq in, twenty roller trips were found advantageous (48,000 ft lb per cu ft of compactive effort). The 48,000 ft lb per cu ft field compaction appears to fall about on an interpolated 48,000 ft lb per cu ft laboratory compaction in Fig. 1; hence, it appears that the relationship between field and laboratory compactive efforts here is about 100% .