PROGRESS REPORT ON DEVELOPMENT AND USE OF STRENGTH TESTS FOR SUBGRADE SOILS AND FLEXIBLE BASE MATERIALS

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SYNOPSIS

This paper stresses the fact that the results of strength tests on soil materials are strongly influenced by inherent quality, density, dry curing, and confinement during absorption. A tentative triaxial compression test procedure has been developed on the basis of studies pertaining to these factors. This procedure employs simplified equipment, thereby making it possible to perform these tests in nearly any normally equipped soils laboratory. The procedure shows promise of great usefulness in the solution of problems relating to subgrades and layered systems of sub-bases and flexible bases.

This progress report on the development of a strength test for subgrade soils and base materials is presented in the hope that the principles of the procedure and the simplified testing equipment described herein may be of assistance in the work of other investigators. The method is not considered to offer a complete and final solution to the depth of base problem. The procedure to be described is particularly applicable to the testing of disturbed materials; however, the triaxial compression portion may be adapted to the testing of undisturbed samples.

The development of a strength test has been based on the premise that the conditions and influences which produce important effects on the load carrying capacity of actual structures in the field, should be reproduced or simulated in any valid laboratory tests procedure. This principle is important whether applied to a laboratory test procedure or large scale field test sections which are intended to check the validity of laboratory tests.

In the following pages, the factors which we are convinced should be given more than the usual attention are described under the general headings: *Inherent Quality*, *Density*, *Dry Curing*, and *Confinement During Absorption*.

DISCUSSION OF THE FACTORS

Inherent Quality—The innate quality of disturbed materials is the most important factor involved and alone is sufficient for a complete study. Fortunately, considerable information has been published on this subject which may be used for this discussion. The introduction of the soil constants to this country by Dr. Karl Terzaghi and the Public Roads Administration was one of the major contributions to the highway branch of soils engineering. These tests are extremely useful but strength tests appear to be necessary, in order to further investigate the effects of densification, moisture control, coarse aggregates and the characteristics of layered soil systems.

Density—The importance of density is stressed in numerous articles on this subject. Results of strength tests shown in Tables 1 and 2 indicate clearly that additional compaction of sandy soils and flexible base materials is accompanied by increased strengths. However, there is justification for concern about the under or over compaction of clay soils as will be shown later in Paragraph 8 of this subject. Although considerable information on this subject is available, it is difficult to devise a laboratory method of compacting specimens for strength tests which will represent the probable conditions obtained under present construction practice and at the same time be susceptible of controlled variation in the orderly investigation of possible improvements in construction practice.

The results from studies of compaction procedures for strength tests have lead to the following conclusions:

1. An impact method for compacting

specimens is preferred in lieu of direct compression methods, because it appears to produce specimens that are more homogeneous.

2. The standard Proctor procedure specified that the moisture-density relations shall be determined by reworking a single soil sample. In cases where materials consist of soft aggregate particles or clay soils, this procedure produces results that are difficult to duplicate with test specimens molded from individual portions of a sample. This is because the soft aggregate particles break down into smaller sizes and the clays form dense residual lumps as a result of the reworking. Since strength test specimens are not made from reworked material, it is believed that moisture-density specimens, intended for guidance in making strength test specimens, should not be reworked but should be made from individual portions of the sample.

3. The total material should be used in making density and strength tests whenever possible. In some cases, where base materials contain very high percentages of coarse aggregate, it may be impossible to obtain densities in the minus ¼-in. portions that will be equivalent to 90 percent of the standard Proctor maximum densities determined for these portions alone, regardless of the amount of compaction applied on the total materials. It also appears that the presence of coarse aggregates increases the strength of well graded flexible base materials.

4. In making strength test specimens the thickness of compacted layers should be given careful consideration. Naturally, where materials containing coarse aggregate are to be tested, the thickness of each layer should be at least as great as the diameter of the maximum size aggregate. Since most standard flexible base materials specifications require that all material shall pass the 2-in. screen, 2 in. has been selected as the maximum aggregate size for testing purposes. When such materials are to be tested it is necessary that each layer be at least 2 in. in depth.

5. In cases where the standard 5½-lb Proctor hammer with a drop of 12 in. was used for compacting some granular materials and clay soils, it was found that the usual thickness of compacted layer (1.5 to 2 in.) was too great to permit uniform compaction throughout each layer. The modified AASHO hammer, weighing 10 lb, and having an 18-in. fall will produce reasonably uniform density throughout a 2-in. layer. Therefore, the use of the heavy hammer is preferred for making strength test specimens. In the case of clean sands of uniform particle size, the proper method of compaction is not known. If it is possible to compact specimens in thin layers, the procedure may be modified to permit the use of the standard Proctor hammer.

6. With such materials, experience indicates that for general practical use, a 6-in. diameter of specimen is the minimum that should be used. Also, it appears that for all materials not highly frictional, a specimen height of 8 in. is adequate. However, for highly frictional materials, this height may introduce some error in test results. The effects of height have not been fully investigated but so far this error appears to be small.

7. To facilitate expression and comparison of these two methods of compaction, compactive effort has been defined in ft-lb of energy per cu in. of specimen. By this means, the compactive energy exerted by the modified hammer may be expressed in terms of the standard Proctor compactive effort which is equal to 6.56 ft-lb per cu in., when the standard, 4-in. diameter, Proctor specimen is assumed to be 5 in. in height before trimming. Therefore, in compacting a 2-in. layer of a 6-in. diameter specimen, 25 blows of the modified AASHO hammer corresponds to one standard Proctor compactive effort. Identical compactive efforts do not necessarily produce identical densities when the efforts are applied with different compaction devices to the same thickness of layer. The comparison of these compactive efforts is misleading with regard to the density produced in specimens unless the depth of each compacted layer is adjusted properly. With a given compactive effort, the heavy modified AASHO hammer, applied to layers 2 in. in thickness, produces greater densities than are produced by the standard Proctor hammer applied to layers of 1½-in. thickness. It was found from trial that similar soil densities were produced when equal compactive efforts were applied by the standard Proctor hammer to layers approximately ¾ in. thick and when applied by the modified AASHO hammer to layers 2 in. thick.

8. For future investigations involving nonswelling soils and flexible base materials, experience indicates that a compactive effort equivalent to approximately twice that employed in the standard Proctor test should be used. This is necessary in order to overcome the friction of the aggregate that resists adequate densification of the minus ¼-in. portion. This conclusion is based on the following: In a limited field investigation, samples consisting of aggregate bearing flexible base materials were secured from a number of construction projects as completed. The density and moisture content of the material in place was determined. In the laboratory, the total material samples were remolded at various compactive efforts and moisture contents. The compactive effort that correlated with the usual construction practice was found to be approximately twice the standard Proctor effort. It has been observed repeatedly that molding of flexible base materials at twice the standard Proctor compactive effort seldom produced densities in the minus ¼-in. portion that were greater than 95 percent of the standard Proctor maximum density determined for this portion alone. The requirement of 95 percent standard Proctor maximum density in the minus ¼-in (or No. 4 sieve) portion is often a part of compaction specifications. In some cases it may be found to be feasible and profitable to compact such materials to higher densities.

Where swelling clays from the upper layers of subgrade are to be tested, lower compactive efforts should be used. Usually strength test specimens consisting of medium to high swelling clays should be molded at a density which is comparable to that produced with the modified hammer, in 2-in. layers and at a compactive effort equivalent to six tenths of one standard Proctor effort as defined in Paragraph 7. At first glance this compactive effort may appear to be too low but in working with clay soils this procedure was found to produce initial densities slightly in excess of those obtained by the standard Proctor procedure. Specimens consisting of swelling clays molded at the optimum moisture content for the compactive effort proposed, and not permitted to dry prior to capillary wetting, usually produce low amounts of swell. Additional compaction does not result in greater subsequent strengths after saturation unless such specimens are loaded during wetting with surcharges much heavier than the weights of pavements. This is because the swelling action destroys the high initial strengths of such dense specimens. If soils are compacted at optimum moisture and maximum density for lower compactive efforts the subsequent swell will be very low, but the subsequent strengths also will be low. Our studies of swelling clays indicate that at the time of sealing or paving the control of moisture content is equally as important as the control of density in order to obtain a subsequent low swell, high strength condition. The data shown in Figures 1 and 2 were obtained by use of the swell test procedure proposed by Allen and Johnson.[1] The following modifications were made:

1. Standard Proctor compaction equipment in lieu of static loads was used for molding specimens.
2. Specimen heights were 2½ in. instead of 2 in.
3. Strength tests were made after completion of swell by use of the Texas modified bearing value punch test. These values are expressed as lb per sq in. required to penetrate the specimen to a depth of ¼ in. with a circular foot having an end area of 0.05 sq in.

The data in these figures indicate that low swell, high strength conditions for upper layers of subgrade can be obtained when the moisture contents at time of sealing or paving range from 5 percent below to 1 percent above that of the standard Proctor optimum moisture content, providing the densities are not too high or too low. Figure 2 indicates that for some high swelling clays, the standard Proctor density is about the minimum density at which major subsequent strengths can be expected.

It may be of interest to note that subsequent swell and strength of a given soil specimen cannot be anticipated unless both density and moisture content at the beginning of capillary wetting are known. For example, consider test results obtained under given conditions for soil in Figure 1. Assume that

[1] Harold Allen and A. W. Johnson, "The Results of Tests to Determine the Expansive Properties of Soils", *Proceedings*, Highway Research Board, Vol. 16, (1936).

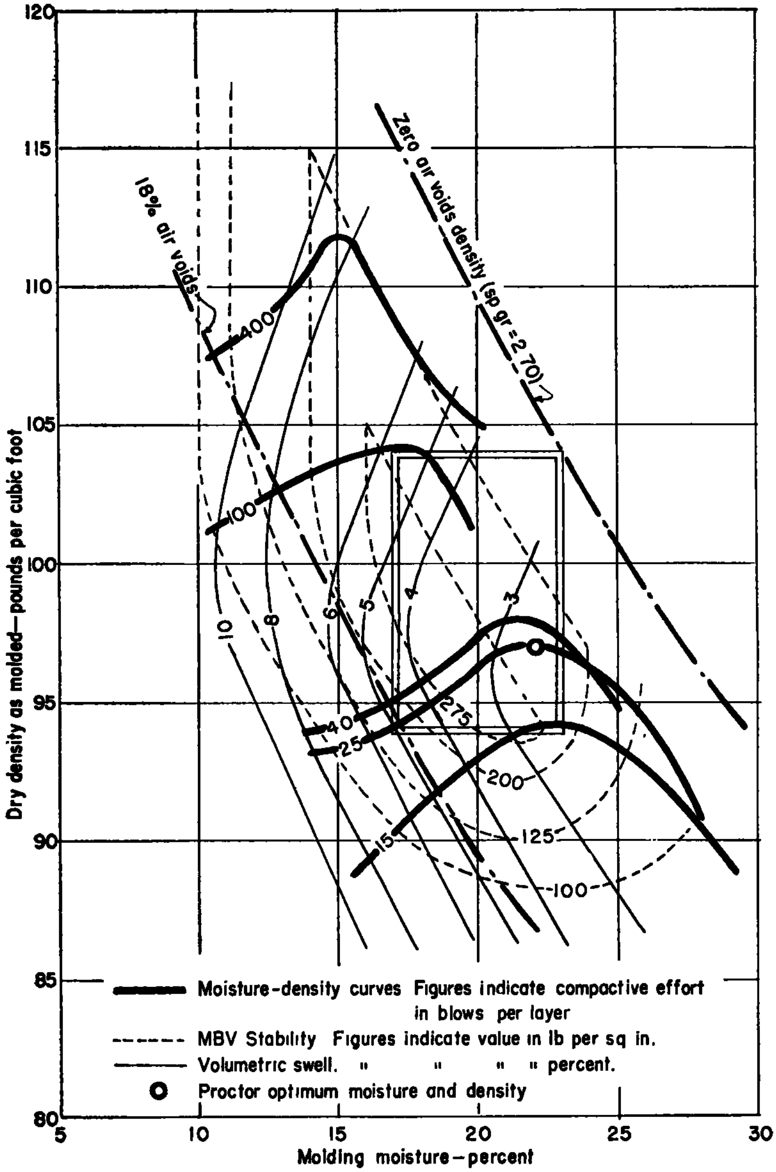

Figure 1. Swell and Strenth Test Results for Clay Soil 39-59-MR (LL = 52; PI = 31)

Specimens were molded at the various moisture density relations shown and subjected (as molded) to moisture, until absorption ceased, under a surcharge equivalent to the weight of six inches of pavement.

this soil is compacted to a density of 95 lb per cu ft, and that the moisture content at the time of compaction and exposure to capillary wetting is 12, 20, or 26 percent. If the original moisture content is 12 percent, the final result will be high swell (10%) and low strength; if original content is 20 percent, the result will be low swell (3.3%) and high strength; if original content is 26 percent, the result will be low swell and low strength.

The intermediate moisture content (20%) results in a final condition that is far superior to the other two. Likewise, when specimens of this soil were subjected to capillary wetting at a moisture content of 20 percent, and at densities of 86, 96, and 104.5 lb per cu ft, may be noted that the percentage of air voids alone does not always reflect the resultant swell or strength of clay soils.

Our studies on medium to low swelling clays are limited but, at present, strength test specimens consisting of these soils are

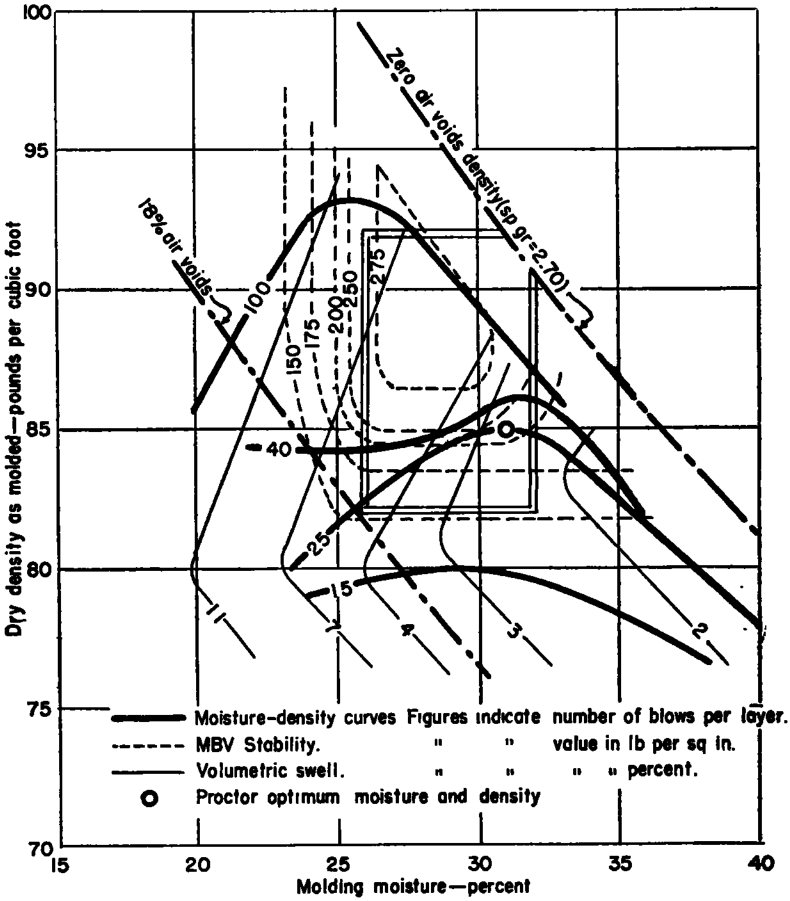

Figure 2. Swell and Strength Test Results For Clay Soil 39-11-MR (LL = 74; PI = 45)

Specimens were molded at the various moisture density relations shown and subjected (as molded) to moisture, until absorption ceased, under a surcharge equivalent to the weight of six inches of pavement.

the subsequent volumetric swells were 6, 3, and 4 percent, respectively. In this case the specimen molded at the intermediate density produced the greatest strength. Therefore, it is not always practical to compact swelling clays from the upper layers of subgrade to extremely high densities in an effort to obtain maximum subsequent strengths. Also, it being molded at a compactive effort equivalent to one standard Proctor effort. This compactive effort for molding specimens ranges in between that used for high swelling clays and granular or non-swelling soils.

Dry Curing—Most investigations made to date, involving strength tests on laboratory specimens or test sections fail to recognize any increased strengths caused by the curing methods used prior to wetting the specimens or sealing the test sections. When members of our field forces first called this phenomenon to our attention several years ago, we were skeptical of its importance except as a means for temporary hardening of the base just prior to application of surfacing. Research studies showed that all moisture removed by drying usually was replaced in time by capillarity, etc., and it seemed that no additional final strength would accrue from such a curing procedure. Nevertheless, some of our field forces contended that permanent effects did occur and that some few so-called "green" bases, consisting of commonly accepted good material were stable when surfaced, but failed to remain stable. It was further contended and demonstrated that some such bases when scarified, recompacted, and properly dry cured before surfacing, served as satisfactory bases thereafter. These experiences focused the laboratory's attention on this factor which seems to be more or less neglected by other investigators. It is highly desirable to develop a laboratory test procedure that measures the final strengths of subgrade and base materials as accurately as possible. In the investigations leading toward the development of such a procedure, the test results have confirmed the importance of the effects of dry curing on final strength. This factor must be reckoned with for the bulk of the work to be done in our part of the country. It is realized that the advantage of this phenomenon cannot be utilized to full extent on projects built under extremely humid or war-time emergency conditions, but these adverse conditions may occur in only a small portion of our peace-time program. Tests made for the design and control of such unusual projects probably should be made without consideration of the drying factor. Inasmuch as strength tests probably will be used as a means for determining the quality and depth requirements for subgrade and base materials, it is highly desirable that the properties measured in the tests be as nearly consistent with those developed by good, sound construction methods as practicable. Any great deviation from this ideal may delay or even defeat the development of satisfactory strength test procedures.

In the laboratory investigation of the above described phenomenon unconfined compression tests were performed on crushed stone-soil flexible base material mixtures, as shown in Table 1. As normally expected for the specimens made at optimum conditions, the moisture removed by drying was taken back into the specimens during capillary absorption. This being the case, it is difficult to deduce by logical reasoning that such great percentage increases in strengths as are shown in the last column of Table 1 can exist. Perhaps previous ideas have been based too greatly on moisture content alone which fails to measure all of the factors involved. It may be noted that strengths obtained from dry cured-wetted specimens of good flexible base materials may be as much as 2 to 2½ times those obtained on duplicate specimens that were not dried prior to capillary wetting. Also, it may be noted that specimens made of a given material at a relatively high compactive effort are somewhat less affected by this phenomenon than are specimens made at lower compactive efforts. This investigation was completed prior to the development of the compaction procedure proposed elsewhere in this report. Specimens 6 in. in diameter and approximately 6 in. in height were molded in three layers at either 110 (2 × std. Proctor effort) or 165 (3 × std. Proctor effort) blows per layer of the standard Proctor hammer (5½ lb dropped 12 in.). The soil constants and gradation of the materials tested are shown in Table 3.

Somewhat the same increases in strength due to dry curing were obtained for modified bearing value tests (MBV defined at bottom of Table 2) made on minus 40-mesh soils, as shown in Table 2. At a glance, it appears that the strengths of these soils were increased by dry curing to a lesser degree than were the materials consisting of crushed stone and soil shown in Table 1. Such a conclusion may be unsound, because two entirely different test procedures and degrees of drying were used in performing tests whose results are shown in these tables. Specimens shown in Table 2, consisting of soil fines, when tested for the dried-wetted stability were dried almost completely before capillary wetting. It is possible that a lesser degree of drying might have produced different stabilities from those obtained. The dried-wetted specimens whose test results are

TABLE 1

Soil: Chapman crushed rock plus various soil binders — Specimen no.	Soil binder %	Soil binder PI	Compaction data: Compactive effort × Proctor	Compaction data: Optimum moisture	Compaction data: Molding moisture	Compaction data: Dry density as molded	Curing data moisture content: When subjected to capillarity	Curing data moisture content: After capillary absorption	Ultimate unconfined compressive strength: Tested as molded	Ultimate unconfined compressive strength: Tested after capillarity no curing	Ultimate unconfined compressive strength: Tested after drying and subjecting to capillarity	Percent strain at ultimate strength	Molded wetter or drier than optimum moisture	Strength of dry-cured wetted specimens expressed as a percentage of the strength of Non-dried wetted specimens
				%	%	lb per cu. ft.	%	%	lb per sq. in.	lb. per sq. in.	lb. per sq. in.	%	%	%
C-32	16	14	2	7.20	7.54	132.04	Not		109			3.8	0.34 Wet	
C-151	16	14	2	7.20	7.31	133.87	7.31	7.36		122		4.8	0.11 Wet	91.0
C-52	16	14	2	7.20	7.32	129.34	3.96	7.43			100	2.7	0.12 Wet	
C-124	16	14	2	7.20	7.60	133.24	4.04	7.76			122	3.3	0.40 Wet	
C-57	16	14	3	7.20	7.40	134.75	Not		125			5.1	0.20 Wet	
C-56	16	14	3	7.20	6.81	132.74	Not		128			2.6	0.39 Dry	
C-152	16	14	3	7.20	7.36	133.41	7.36	7.68		89		5.0	0.16 Wet	
C-68	16	14	2	7.20	7.51	134.88	3.63	7.25				3.0	0.31 Wet	
C-121	16	14	3	7.20	7.34	134.45	3.87	7.59			172	3.5	0.14 Wet	185
C-116	16	14	3	7.20	7.12	134.64	4.73	7.44			144	3.8	0.08 Dry	
C-29	20	11	2	7.45	7.13	134.99	Not		95			4.5	0.32 Dry	
C-153	20	11	2	7.45	7.55	135.03	7.55	7.68		64		5.5	0.10 Wet	175
C-51	20	11	2	7.45	7.59	133.00	4.03	7.40			112	3.5	0.14 Wet	
C-60	20	11	3	6.66	6.73	135.46	Not		160			4.1	0.07 Wet	
C-154	20	11	3	6.66	6.79	135.46	6.79	7.06		111		3.7	0.13 Wet	
C-69	20	11	3	6.66	6.88	133.96	3.59	7.11			138	2.5	0 22 Wet	132
C-127	20	11	3	6.66	6.98	135.86	3.80	7.40			156	3.1	0.32 Wet	
C-16	25	9	2	7.25	7.63	135.88	Not		92			6.6	0.38 Wet	
C-15	25	9	2	7.25	7.06	135.79	Not		104			4.5	0.19 Dry	
C-155	25	9	2	7.25	7.23	136.49	7.23	7.28		57		4.9	0.02 Dry	
C-133	25	9	2	7.25	7.07	133.18	3.84	7.39			140	3.0	0.18 Dry	257
C-45	25	9	2	7.25	7.49	136.74	3.44	7.46			153	3.4	0.24 Wet	
C-19	25	9	2	7.25	7.76	134.71	4.07	7.80			103	5.0	0.51 Wet	
C-65	25	9	3	6.75	6.62	137.24	Not		165			3.9	0.13 Dry	
C-63	25	9	3	6.75	6.88	137.42	Not		148			4.1	0.13 Wet	
C-156	25	9	3	6.75	7.21	137.37	7.21	7.22		85		5.5	0.46 Wet	198
C-71	25	9	3	6.75	6.71	137.36	3.47	6.71			168	2.5	0.04 Dry	
C-104	25	7	3	6.25	6.51	137.51	Not		173			3.3	0.26 Wet	
C-110	25	7	3	6.25	6.47	137.06	3.10	6.52			185	3.3	0.22 Wet	
C-9	30	7	2	7.46	7.34	135.92	Not		103			3.2	0.12 Dry	
C-10	30	7	2	7.46	7.76	135.63	Not		84			2.0	0.30 Wet	
C-161	30	7	2	7.46	7.18	135.84	7.18	7.15		71		4.9	0.28 Dry	
C-157	30	7	2	7.46	7.98	135.64	7.98	7.78		22		2.9	0.52 Wet	190
C-142	30	7	2	7.46	7.47	134.87	3.22	7.36			135	3.0	0.01 Wet	
C-22	30	7	2	7.46	7.55	135.27	4.81	7.10			161	2.9	0.09 Wet	
C-35	30	7	3	6.71	6.85	137.74	Not		158			2.3	0.14 Wet	
C-36	30	7	3	6.71	7.54	136.75	Not		157			3.3	0.83 Wet	
C-158	30	7	3	6.71	6.97	136.88	6.97	6.97		86		5.2	0.26 Wet	
C-139	30	7	3	6.71	6.80	135.94	3.25	6.92			205	2.3	0.09 Wet	148
C-46	30	7	3	6.71	6.81	138.07	3.52	6.51			249	2.3	0.10 Wet	
C-90	30	7	3	6.71	7.56	136.39	3.50	6.81			164	2.9	0.85 Wet	
C-108	30	11	3	7.10	7.56	134.54	Not		131			4.5	0.46 Wet	
C-112	30	11	3	7.10	7.62	133.68	4.38	7.48			143	5.0	0.52 Wet	
C-5	35	6	2	7.40	7.35	134.73	Not		78			3.8	0.05 Dry	
C-11	35	6	2	7.40	7.66	135.22	Not		66			6.4	0.26 Wet	
C-12	35	6	2	7.40	8.49	133.70	Not		28			5.1	1.05 Wet	
C-159	35	6	2	7.40	7.53	135.41	7.53	7.46		60		4.1	0.13 Wet	
C-144	35	6	2	7.40	7.33	133.35	3.25	7.63			141	2.5	0.07 Dry	235
C-23	35	6	2	7.40	7.83	134.67	4.85	7.46			139	2.9	0.43 Wet	
C-100	35	6	2	7.40	8.35	132.51	3.81	7.58			100	3.0	0.95 Wet	
C-40	36	6	3	6.67	6.85	136.40	Not		139			2.8	0.18 Wet	
C-41	35	6	3	6.67	7.35	135.51	Not		133			2.8	0.68 Wet	
C-160	35	6	3	6.67	6.67	135.47	6.67	6.88		128		2.9	— Opt	178
C-48	35	6	3	6.67	6.71	136.59	3.36	6.47			228	1.8	0.04 Wet	
C-92	30	6	3	6.67	7.42	135.06	3.56	6.93			185	2.2	0.75 Wet	

shown in Table 1 were not dried severely, as is indicated by the moisture contents prior to capillary wetting. The soil constants and gradations of the soils used for deter-

TABLE 2

A	B	C	D	E	F	G	H	I	J	Remarks	
Soil lab. no.	Compactive effort (no. Proctor Hammer Blows per layer	Optimum moisture	Molding moisture	Dry molding density	Moisture content when subjected to capillarity	Moisture content after absorption	Stability values MBV[a]: As molded (see column D for % moisture when tested)	Stability values MBV[a]: As molded and subjected to capillarity (See column G for % moisture when tested)	Stability values MBV[a]: After removing molding moist. & subjecting to capillarity (See column G for % moisture when tested)	Specimens 4 in. diameter and 2½ in. height molded in 2 layers	Strength of dry cured-wetted specimens expressed as a percentage of the strength of non-dried wetted specs.
		%	%	lb per cu ft.	%	%					
39-7-MR	15	11.0	10.7	115.4	10.7	11.1	180	225		Molded drier than optimum	118
	15	11.0	10.7	115.1	0.0	10.9	225		265		
	15	11.0	12.6	111.6	12.6	11.9	70	100		" wetter " "	
	15	11.0	12.7	111.3	0.1	10.9	75		260		
	25	9.5	8.9	117.6	8.9	11.3	325	240		" drier " "	173
	25	9.5	9.8	117.5	0 1	10.7	260		415		
	25	9 5	10.7	116.3	10.7	10.5	160	200		" wetter " "	
	25	9.5	11.5	114.6	0.1	9.9	80		398		
	40	9.0	8.5	120.4	9.0	9.9	450	265		" drier " "	185
	40	9.0	8.1	120 8	0.0	9.9	700		490		
	40	9.0	10.2	118.8	10.2	10.3	175	225		" wetter " "	
	40	9.0	10.4	117.8	0.1	10.0	375		372		
	100	8.0	7.6	125.3	7.6	8.5	750	550		" drier " "	136
	100	8.0	7.4	127.2	0.0	9.0	1,200		750		
	100	8.0	9.6	121.8	9.6	9.5	210	300		" wetter " "	
	100	8.0	9.5	122.3	0.1	9.6	500		655		
	400	7.0	7.6	128.6	7.6	8.4	675	675		" " " "	174
	400	7.0	7.4	128.7	1.1	7 1	1,075		1,175		
	400	7.0	6.4	129.1	6 4	8.4	1,075	500		" drier " "	
	400	7.0	6.4	130.5	0 0	8.1	1,350		1,615		
39-10-MR	15	16.0	15.9	105.9	16.0	18.3	350	225		" " " "	124
	15	16.0	15.4	108.0	0.0	17.0	475		280		
	15	16.0	18.9	102.3	18.9	20.1	100	100		" wetter " "	
	15	16.0	18.7	103.1	0.0	18.6	140		145		
	25	14.5	12.1	105.9	12.1	19 0	700	225		" drier " "	
	25	14.5	12.1	106.5	0.2	17.8	700		325		
	25	14.5	15.2	107.6	15.2	17.9	550	275		" wetter " "	131
	25	14.5	14.7	108.2	0.0	17.6	560		360		
	40	14.0	11.8	113.7	11.8	10.4	1,225	700		" drier " "	77
	40	14.0	13.7	112.0	0.0	14 5	775		425		
	40	14.0	14.8	113.1	4.8	15 4	500	550		" wetter " "	
	40	14.0	16.8	107.3	0.0	17.3	250		280		
	100	12.5	11.6	118.1	12.5	13.7	1,700	1,550		" drier " "	
	100	12.5	10.3	114.1	0.5	14.2	1,650		670		
	100	12.5	13.2	117.0	13.2	14.1	1,300	1,000		" wetter " "	108
	100	12.5	12.6	116.4	0.2	14.0	1,325		1,075		
	400	11.0	10.0	121.7	10.0	12.4	3,375	1,420		" drier " "	
	400	11.0	9.1	117.2	0.2	14 5	2,900		765		
	400	11.0	11.4	121.9	11.4	12.6	1,875	1,350		" wetter " "	74
	400	11.0	11.3	119.5	0.3	13.4	2,675		1,000		

[a] The lbs. per sq. in required to penetrate the specimen a depth of ¼ in. with a circular foot having an end area of 0.05 sq. in.
None of above specimens consisting of soil 39-7-MR swelled more than 0.6 percent.
None of above specimens consisting of soil 39-10-MR swelled more than 1.7 percent.

mining the data in Table 2 are shown in Table 3. It is recognized that there are some soils whose strengths are not benefitted by dry curing. It is also recognized that, in the case of highly plastic clay soils, the swelling effects following drying usually overcome the benefits from the dry curing process. It appears that benefits from the dry curing method generally accrue to soils that have plastic indexes below 15, because the swelling and shrinking properties of such soils are small.

The data presented here indicate that any strength test performed on laboratory specimens consisting of granular soils and flexible base materials should recognize the effects of dry curing, unless the project under consideration is to be built under emergency and (or) extreme humid conditions. Also, it is curing variable in order that the behavior of a given material at a given strength could be known. However, many engineers are of the opinion that these tests represent the results to be expected from all of the better types of construction when similar load frequencies occur on similar subgrades and on

TABLE 3
SOIL CONSTANTS AND GRADATIONS

Lab. No.	LL	PI	FME	CME	SL	LS	SR	Class (PRA)	Soil Binder
45-120-E	31	14	25	22	17	8.6	1.85	A-4	16
45-121-E	26	2	24	1	20	0.6	1.64	A-3	100
Comb. 1		11							20
2		9							25
3		7							25
4		7							30
5		11							30
6		6							35
39-7-MR	18	3	15	4	15	1.4	1.82	A-2	93
39-10-MR	28	8	23	12	23	2.8	1.60	A-4	100

Percent Retained on

Lab. No.	Round Opening Screens					Square Mesh Screens						Grain Diameter			Specific Gravity of Soil Binder
	Opening in In.					Sieve Numbers						mm			
	1½	1	¾	½	¼	10	20	40	60	100	200	.05	.005	.001	
45-120-E	11	31	44	55	68	77	81	84	86	87	90	91	96	98	2.67
								0	1	25	87	91	96	96	2.63
Combination 1	11	30	42	53	65	74	78	80							
2	10	28	39	49	61	69	72	75							
3	10	28	39	49	61	69	72	75							
4	9	26	37	46	57	64	68	70							
5	9	26	37	46	57	64	68	70							
6	9	24	34	43	53	60	63	67							
39-7-MR						1	2	7	23	48	70	78	94	98	2.65
39-10-MR								0	3	7	29	49	91	97	2.66

Sample Identification

Lab No.	Identification	Type of Materials
45-120-E	Pit-Chapman Ranch, Williamson Co.	Cr. Limestone
45-121-E	Pit-Elgin, Bastrop Co., Texas	Sand
Comb. 1	100 parts 45-120-E plus 5 parts 45-121-E	
2	100 parts 45-120-E plus 12 parts 45-121-E	
3	Same as Comb 2, except soil binder from Comb. 4	
4	100 parts 45-120-E plus 20 parts 45-121-E	
5	Same as Comb 4, except soil binder from Comb. 1	
6	100 parts 45-120-E plus 29.2 parts 45-121-E	
39-7-MR	Hwy No 73, Austin County	Sandy Soil
39-10-MR	Hwy No. 208, Coke County	Silty Sand Clay

believed that the strength of test sections built for field loading investigations are strongly affected by the drying factor. Numerous extensive investigations of field test sections have been performed without introducing such variables as the drying factor herein described. It is quite possible that these sections should have been built and tested without introducing the dry similar granular subgrade reinforcement or flexible base materials. Such opinions do not give consideration to the possibility that the behavior of these sections could have been quite different if each layer of granular subgrade reinforcement or flexible base material had been dry cured. If these layers had been properly dry cured prior to covering instead of being sealed in a "green uncured" condition, lesser depths of granular materials probably would have been required for wheel load support.

Confinement during Absorption—Since all layers of subgrades and bases have lateral support from adjacent areas during moisture accumulation, it was considered advisable to investigate the influence of this factor on strength tests. It was found that the strengths of some materials were affected greatly by small amounts of lateral pressure during absorption. Our information on this subject is too limited at present to identify the materials that may or may not be affected by this factor. In one case, strength tests made on a sand-clay soil having a PI of 6 showed that the compressive strengths of the specimens wetted under the influence of a 1-lb per sq in. lateral pressure were twice those of specimens that were wetted while unsupported. The effects on strengths produced by various intensities of lateral pressure have not been determined. At present, we are using a lateral pressure of 1 lb per sq in. around the periphery of the specimen during capillary wetting. This is approximately equivalent to the hydrostatic pressure of soil at rest, at a point 12 in. below the surface of the pavement. An approximate vertical surcharge, usually $\frac{1}{3}$ to $\frac{1}{2}$ lb per sq in. also is applied to the specimens.

TRIAXIAL COMPRESSION TEST

A triaxial compression test has been chosen for the following reasons:

1. The test results are in terms that are applicable to many of the equations that have been proposed for use in the solution of soil mechanics problems.
2. Test results are not affected by the restraint of molds.
3. Relatively large size aggregates can be included in the materials to be tested without resorting to extremely large size specimens. This reduces the weight of equipment and materials that otherwise might be required.
4. To date the preliminary test results for quality of materials appear to be in line with field experience.

As a result of the previously described investigations, together with many other experiences with strength tests, a tentative test procedure has been developed. The procedure is not considered to be entirely out of the development stage, and the investigation of application of test results to actual problems is far from complete.

Tentative Test Procedure—The following is a summary of the procedure and equipment proposed. A more detailed procedure is shown in the appendix.

1. 200 to 300 lb of air dried material is separated into the various particle sizes. Clay soil lumps are crushed to pass the $\frac{1}{4}$-in. screen (See Fig. 3).
2. To make individual specimens the components are recombined in exact amounts representing the original material.

Figure 3. Separated Portions of Sample

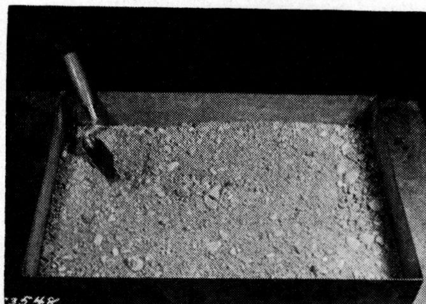

Figure 4. Mixture of Sample Portions

3. A weighed quantity of water which, added to the determined hygroscopic moisture, will provide a chosen final moisture content is mixed intimately with the material. Care is taken to saturate coarse aggregate particles. Materials containing impervious clay lumps are stored overnight to permit equalization of moisture distribution (See Fig. 4).

4. The specimens are 6 in. in diameter by 8 in. in height, compacted in 4 equal layers of 2-in. thickness, each. The total batch described in step 3 is used to form one specimen. By means of the Modified AASHO hammer (See Fig. 5) a selected number of hammer blows[2], depending upon the characteristics of the material and its proposed position in the field, are applied to each layer. The tops of all except final layers are lightly scarified in order to increase the bond between layers. The final, upper surface is carefully levelled (See Fig. 6).

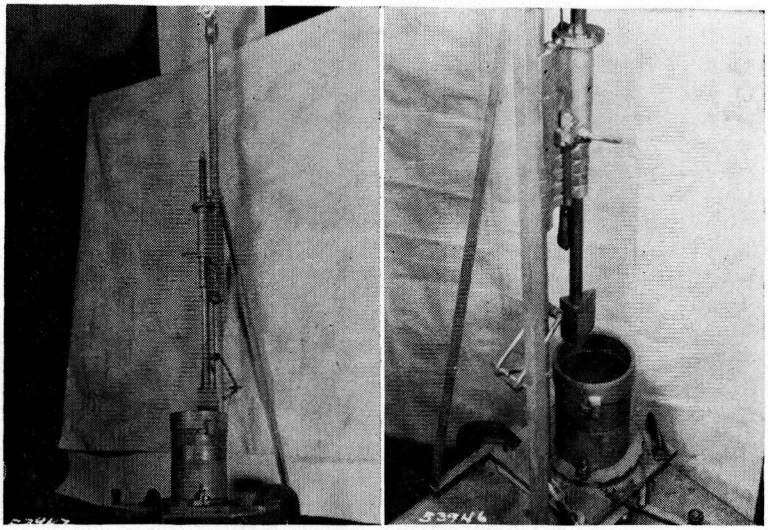

Figure 5. Compaction Equipment

5. The height of the specimen is measured by means of the micrometer dial assembly shown in Figure 7. The diameter of the mold is known. The specimens are weighed. A preliminary estimate of dry density may be made at this stage. If the height varies more than 0.25 in. from the standard, the weight of material used for each specimen is readjusted.

6. A moisture-density curve is determined.

7. Five specimens, as nearly identical as possible, are compacted at the optimum moisture content for the selected compactive effort. (Sets of specimens from the dry side or wet side of the optimum moisture curve, also may be made.) All specimens are measured and extruded from the molds. Porous stones are placed on the top and bottom.

8. All specimens are stored overnight in the moist room. They are protected from free or capillary moisture.

9. Specimens consisting of materials that do not develop shrinkage cracks are partially dried by placing in an air-drying oven (forced draught at 140 F.) for a period of 8 hr. Upon removal, the specimens are allowed to stand overnight in the open laboratory. The specimens again are weighed. Usually, about one-third to one-half of the molding moisture is removed. Materials that tend to develop shrinkage cracks are not dried.

10. The axial cells, deflated by vacuum, are placed on the specimens (See Fig. 8). If the specimens consist of relatively impermeable material, a layer of slitted filter paper, overlapping the upper and lower porous stones, should be wrapped around the specimens prior to placement of the cells. A suitable vertical surcharge (about 0.33 to 0.50 psi for most subgrades) is placed on the top stone. The specimens are then placed in pans of water so that the water level is ½ in. below the bottom of the specimen itself. This assembly is then placed on the storage rack in the moisture room and connected with the constant pressure air

[2] Described more fully in the detailed procedure of the appendix.

manifold. The usual lateral pressure is one psi (See Fig. 9 and 10). (By careful adjustment, properly constructed cells will hold low pressures for some time, without the necessity of the constant pressure device.) The specimens are permitted to absorb

Figure 6. Finishing Surface of Specimen

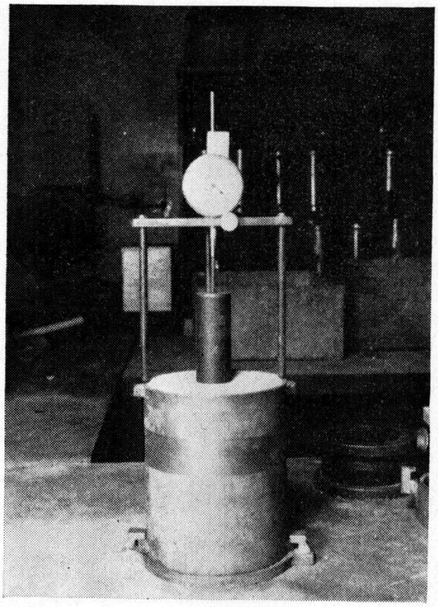

Figure 7. Measuring Height of Specimen

water by capillarity until equilibrium is attained. The time required for capillary equilibrium ranges from a few days to several weeks, depending upon the permeability of the material. At the end of this period the specimens are weighed and measured in order to determine absorption and swell and are ready for testing.

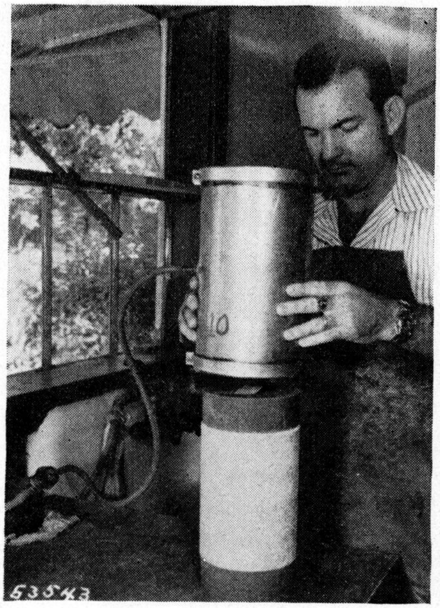

Figure 8. Placing Cell on Specimen

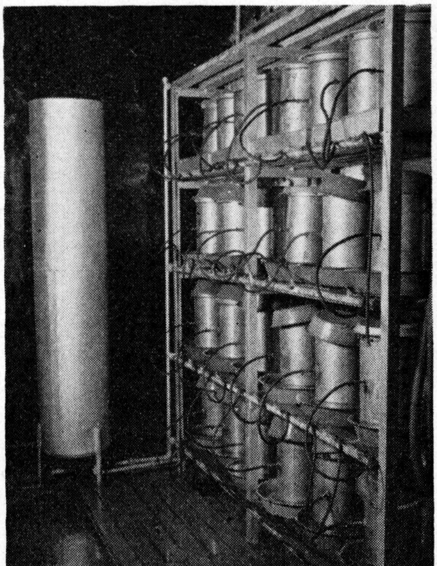

Figure 9. Lateral Pressure System Used in Moist Room During Capillary Absorption

11. Each specimen is tested in compression at a constant lateral pressure. The five identical specimens are usually tested at

lateral pressures of 3, 5, 10, 15 and 20 psi, respectively. All other specimens made for the moisture-density curve are tested at one lateral pressure, usually 5 psi. This pressure is applied by means of the cells, supplied by an auxiliary air tank. (See the schematic drawing in Figure 11.) Any suitable press

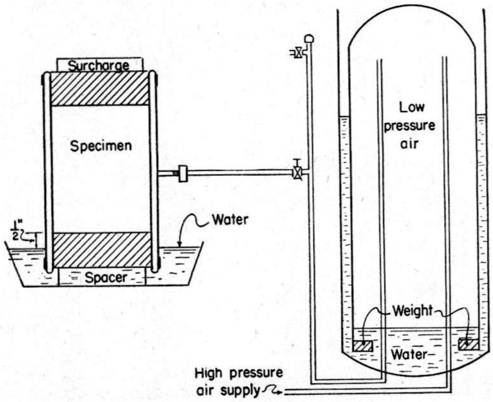

Figure 10. Diagram of Equipment Used During Capillary Wetting

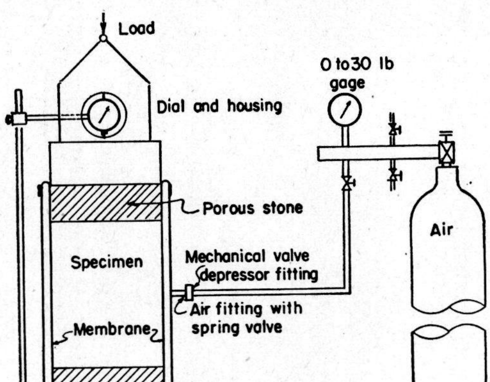

Figure 11. Diagram of Triaxial Compression Test Assembly

may be used. Figure 12 shows a Southwark-Emery press and Figure 13 shows a shop-made, gear-jack press in which the load is measured by a proving ring. Deformation is measured by a micrometer dial mounted along the central axis of the specimen. The rate of strain is 0.15 in. per min. Simultaneous readings of load and deformation are taken

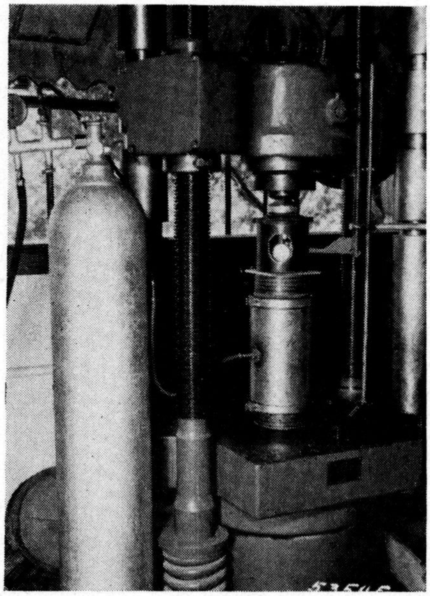

Figure 12. 200,000-lb Southwark Emery Testing Machine

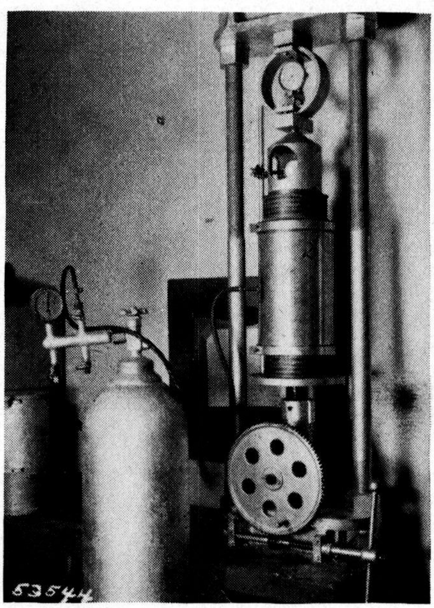

Figure 13. Gear Jack Assembly Press

at intervals of 0.01-in. deformation. Loading continues until the specimen fails.

12. After the completion of the compression test, the entire specimen is dried at 110 C. On the basis of the total dry weight, extra

data as to density, moisture content, moisture absorption, etc., may be calculated.

13. From the test data, stress-strain curves are plotted as shown in Figure 14. From the principal stresses at the instant of failure, Mohr's diagram of stress is constructed. Values of cohesion and the angle of internal friction are determined as shown in Figure 15.

APPLICATION OF TEST RESULTS

Inasmuch as this test procedure has been developed recently, the application of test results to actual problems is only in the formative stage. However, enough of these tests on materials of known behavior are available to indicate the following:

1. All of the untreated soil materials that have been used successfully in the construction of final courses of flexible bases, when tested in the above manner, have cohesion values of at least 12 psi and angles of internal friction in excess of 30 deg.

2. The test results of all other natural soil materials, that have not been satisfactory as final base courses, have not been within the above limits.

3. Changes of plastic index, grading, or density are directly reflected in the results of the triaxial compression tests in an orderly and apparently logical manner.

An attempt is being made to apply these test results to the depth of base problem by means of the elastic theory. Since the values of cohesion and friction are based on stresses at rupture, and since it is undesirable to design against complete failure, it seems to be necessary to apply a factor of safety. The determination of the elastic limits of soils from their stress strain curves lacks definition and accuracy. Therefore, a blanket factor of safety, based upon experience, and arrived at by a proportionate reduction in all indicated shearing strengths along the envelope of rupture, seems to offer the most promise. It should be noted that the factor of safety that applies to most soils does not appear to be satisfactory for highly frictional, cohesionless materials and that such material

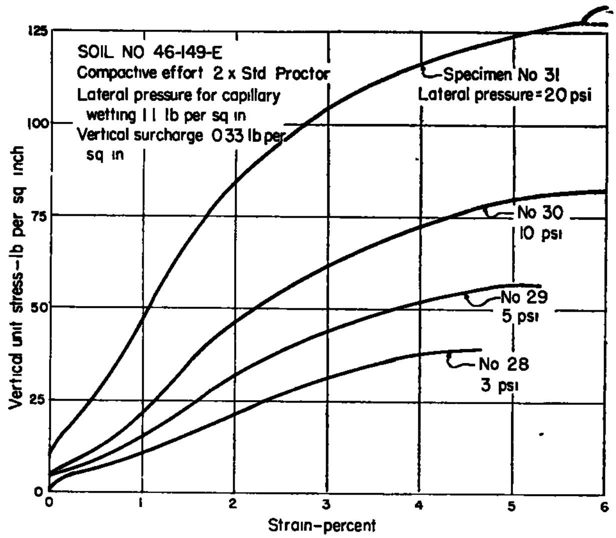

Figure 14

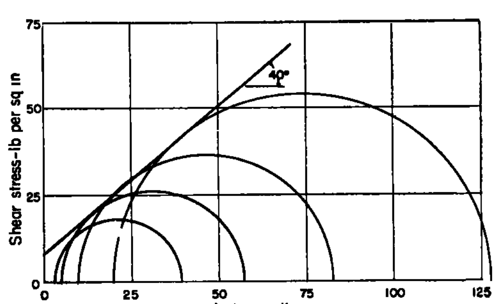

Figure 15

will require special methods of interpretation. The study of the application of the test results is so incomplete that further comment is not justified at present.

SUMMARY

The factors believed to exert major influences on the strengths of subgrade and flexible base materials, such as inherent quality, density, dry curing, confinement, etc., have been studied on the basis of field and laboratory results. It is believed that these factors are ever present regardless of how they are evaluated by tests, and that their importance cannot be overemphasized to those who are engaged in laboratory or field strength tests leading to a practical depth of base design method. It is believed that considerable confusion in interpreting results of such tests can be avoided by recognizing these variables. On the basis of our studies of tests for the determination of the strength of soil materials, the Materials and Tests Laboratory of the Texas Highway Department has attempted to devise a test method whereby these major influences will be properly evaluated. Our efforts have fallen short of perfection and for this reason, comments or suggestions will be helpful and appreciated.

ACKNOWLEDGEMENTS

Since ideas from the entire field of soil mechanics have been incorporated herein, it is impracticable for the author to make specific acknowledgement of indebtedness to all of the persons concerned. Within our organization, acknowledgement is made to: Mr. R. J. Hank, Engineer of Materials and Tests and Mr. A. C. Love, Senior Laboratory and Research Engineer, for their guidance and encouragement; to Mr. W. H. Moore, Soils Engineer, and Mr. J. E. Hunt, Junior Soils Engineer, for the development of testing procedures and simplified testing equipment, and for assistance in preparing the report; to Mr. Homer A. Meier, Soils Engineer, for the investigation of the swelling characteristics of soils and the MBV tests; to Mr. Frank H. Scrivner, Junior Soils Engineer, for his studies of the application of the elastic theory to the interpretation of test results; and to numerous other members of the Materials and Tests staff for the actual performance of contributory investigations.

APPENDIX

TRI-AXIAL COMPRESSION TEST PROCEDURE FOR SOILS AND FLEXIBLE BASE MATERIALS

I. Apparatus

Large oven, for air drying, forced draught, controlled to 140 F

Large oven, for drying, controlled to 110 C

Pans, large capacity, for slaking (Fig. 7, text—background)

Pans, wide, shallow, for mixing (Fig. 4, text)

Screens & sieves, full set from 2 in. to No. 40

Scales, rated capacity 30 lb, overload capacity 60 lb, calibrated to 0.01 lb

Water jar, capacity approx. ½ gal, with sprinkler top

Compaction hammer, string and pulley operated; weight of ram—10 lb; controlled fall of ram—18 in. (modified AASHO); striking face of ram—40 deg segment of 3-in. radius circle—area of face—3.1416 sq in.; ratchet operated turntable to rotate mold for spacing of ram blows; cylindrical spacer block with hand hold, approx. height—3 in.; and required diameter to provide sliding fit in compaction mold (Fig. 5 and 6, text)

Compaction mold with removable collar, to fit turntable; nominal inside diameter—6 in; nominal height—8 in.; wall thickness—¼ in. ± $\frac{1}{16}$ in., reamed and ground true and smooth inside (Fig. 5 and 6, text)

Porous stones, two required per specimen; height—2 in.; diameter to provide sliding fit in mold; standard specifications: Blank-A-100-Blank-Z-12-V (Fig. 8, text and Fig. 16)

Measuring device, for determining exact height of specimen—micrometer dial assembly (Fig. 7, text)

Axial cell, one required per specimen; a light weight, moderately rigid, metal cylinder; inside diameter 6¾ in.; fitted with a standard air valve; inside of the cylinder is a tubular, flexible membrane; nominal diameter—6 in.; folded outward over the ends; cemented and clamped on outside upper and lower edges; upper and lower retainer rings with inside diameter to clear the porous stones, mutually held in place by two turnbuckles, are provided for use when the applied lateral pressure is high (Fig. 8 and 9, text and Fig. 16)

Aspirator or other simple vacuum pump (Fig. 8, text)

Capillary tank or individual pans and storage racks (Fig. 9, text)

Constant pressure air supply with manifolds —optional (Fig. 9 and 10, text)

Press, low capacity, to eject specimens from the mold

Press, for compression test; minimum capacity for untreated soils—10,000 lb, for chemically treated soils—30,000 lb; controllable rate of strain; independent weighing mechanism with sensitivity of 25 lb or less (Fig. 13, text)

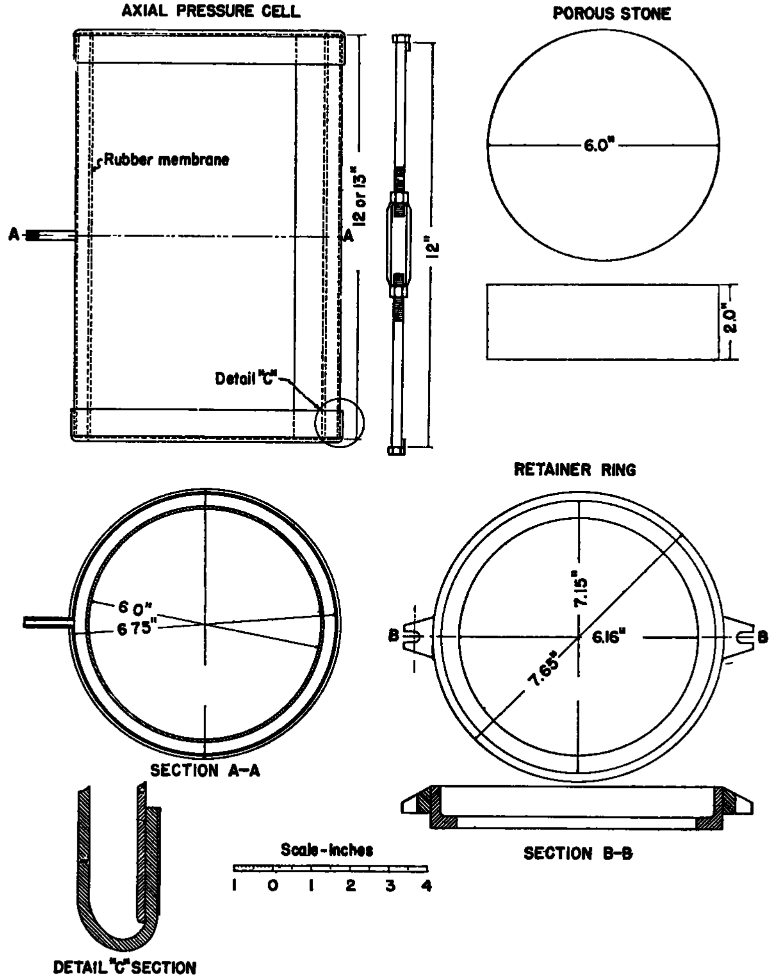

Figure 16. Axial Pressure Cell Assembly

Auxiliary compressed air storage tank with suitable pressure guages, valves, and air lines with standard and valve depressor fittings (Fig. 12 and 13, text)

Air compressor or suitable pump

Micrometer dial, calibrated in 0.001 in., to measure deformation of test specimens (Fig. 12 and 13, text)

Auxiliary equipment: deformation dial support; dial housing to transmit load while permitting view of dial; cylindrical loading blocks; spherical loading block (Fig. 12 and 13, text)

Miscellaneous equipment: small tools,

trowels, soil scoops, drying pans, spatulas, brushes, thermometers, tamps, etc.

II. Quantity of Sample

Secure an adequate quantity of sample, representative of the material to be tested—usually, 200 to 300 lb of dry material are required for each compactive effort to be investigated. The dry weight of individual specimens will range from about 11 lb for plastic clay soils to about 18 lb for well graded base materials. The number of specimens for each compactive effort will range from 11 to 21, estimating six specimens for the moisture density curve and five identical specimens at each molding moisture content to be investigated.

TABLE 4

Sieve	Cumulative	Component	Dry Wt. Component
	Retained		
	%	%	*lb.*
1½-in.	10	10	1.8
1-in.	30	20	3.6
¾-in.	45	15	2.7
½-in.	50	5	0.9
¼-in.	60	10	1.8
No. 10	65	5	0.9
No. 20	72	7	1.26
No. 40	75	3	0.54
	Passing		
No. 40	25	25	4.5
Totals	100	100	18.0

III. Preparation of Sample

Spread out the sample on a smooth working surface where it will be air-dried by sun and wind. Turn and rake the material to insure uniform drying. If outdoor drying is impracticable, a forced draught oven in which the temperature does not exceed 140 F. may be employed. Reduce the sample to the necessary quantity by quartering. From this point the detailed procedure should be adjusted to the characteristics of the material.

A. Soils Containing Aggregate

Crush or remove aggregate larger than 2 in. in accordance with the procedure contemplated in construction.

1. When relatively precise methods are required to maintain acceptable uniformity in grading of individual specimen:

Specimens made of flexible base materials which contain more aggregate than binder tend to vary considerably in density, structure, strength, etc., if the grading varies. This variation may be eliminated by reconstructing each specimen from its component parts. Slake the air dried material and separate the binder from the aggregate by washing through the No. 40 sieve. Again air dry both portions. Determine the hygroscopic moisture content of the soil binder and calculate its total dry weight. Store the binder in covered containers to prevent moisture change. Usually, it is safe to assume that the air dried aggregate contains no hygroscopic moisture. Separate the washed aggregate into several particle sizes by screening (Fig. 3, text). Determine weights and calculate the grading as usual, but also express the grading as component percentages. Estimate the total weight of material required for one specimen.

Example

Specimen: Diam. = 6.20 in.; Height = 8.00 in.; Approx. vol. = 0.1335 cu ft;

Compacted dry density (estimated) = 135 lb per cu ft;

Wt. specimen, oven-dry = 0.1335 × 135 = 18.02, say 18 lb

From the grading data in Table 4 calculate the weights of the various particle sizes to be combined:

From the oven-dry weight and the hygroscopic moisture content, calculate the air-dry weight of the soil binder portion:

Hygro. moist. = 3 percent; Oven dry wt. = 4.5 lb

Air dry wt. = 1.03 × 4.5 = 4.64 lb

Estimate, or from previous data select, the moisture content for compaction, and calculate the weight of water required:

Dry wt. specimen = 18 lb; Desired moisture content = 9 percent

Dry wt. soil binder = 4.50 lb; Hygro. moisture = 3 percent

Total water required = 0.09 × 18 = 1.62 lb

Hygroscopic water = 0.03 × 4.5 = 0.14 lb

Net wt. water required = 1.62 − 0.14 = 1.48 lb

Weigh the calculated, net quantity of water into a jar having a sprinkler top.

In mixing the water and the material it is essential that the aggregate be thoroughly saturated and, in the case of plastic soil binder, that the moisture be uniformly distributed through the binder. Failure to meet these conditions will produce false, erratic test results. The following standardized mixing procedure is recommended:

Weigh out all of the plus ¼-in. aggregate and spread in the large mixing pan. Sprinkle with water from the weighed supply and mix with a small trowel. Continue the addition of water and the mixing until a film of free water remains on the aggregate. Weigh the minus ¼-in., plus No. 40 aggregate into a small enameled pan and saturate with water from the weighed supply. Transfer this saturated, small aggregate to the large mixing pan and mix with coarse aggregate. Weigh out the required amount of binder soil and add a thin layer of binder to the material in the large pan. Sprinkle with water from the weighed supply, and mix with cutting, lifting, and turning strokes of the trowel. Avoid compressing the material. Continue additions of binder and water until all material and water have been thoroughly mixed (Fig. 4, text). Allow the mix to stand so that the moisture will further disperse itself; 15 to 30 min standing will be sufficient when the binder soil is very permeable; overnight standing with suitable protection against moisture loss or gain will be required when the binder is a clay soil of low permeability.

2. When the material can be divided by quartering into portions having acceptable uniformity:

By careful quartering, divide the sample into portions of convenient size and store in covered containers. Determine the hygroscopic moisture content of a representative portion of the total material. Estimate the weight of air dried material required for one specimen as described in III., A., 1 above, except that the hygroscopic moisture content will apply to the total specimen. Weigh out the exact amount of material required, taking care to maintain the grading as well as possible. Separate the material into coarse and fine portions with the No. 10 sieve. Estimate the quantity of water required for compaction as described in III., A., 1, except that the hygroscopic moisture content will apply to the entire specimen. Weigh the required amount of water into a jar with a sprinkler top. The mixing should be performed as described in III., A., 1, considering the material retained on the No. 10 sieve as coarse aggregate, the material passing the No. 10 sieve as binder, and omitting the individual treatment of the small aggregate. The mixed material should be allowed to stand for a period of time as described in III., A., 1 before being compacted.

B. Soils Without Aggregate

Crush the clods to pass the ¼-in. screen or the No. 10 sieve (the less permeable soils require the smaller size). Mix the air dried sample thoroughly and store in covered containers to prevent moisture change. Determine the hygroscopic moisture content of a representative portion. Estimate the weight of air dry soil required for one specimen as described in III., A., 1. Weigh out the exact amount of air dry soil. Separate this material into coarse and fine portions with the No. 10 or the No. 40 sieve. Calculate the amount of water required for compaction as described in III., A., 1 and weigh this quantity of water into a jar with a sprinkler. The mixing should be performed as described in III., A., 1, treating the material retained on the No. 10 or the No. 40 sieve as the coarse aggregate and the material passing the sieve as the binder, and omitting individual wetting of intermediate particle sizes. If lumps or balls are formed during mixing they should be broken by forcing through a No. 4 or No. 10 sieve and remixed with the sample. After mixing the material should be allowed to stand as described in III., A., 1. This is particularly necessary in the case of plastic clay soils. After standing, the material will be ready for compaction.

IV. Compactive Effort

Definition: Compactive effort is defined as the total energy, expressed in ft-lb, delivered by the compaction hammer to each cu in. of specimen.

When, as proposed in this procedure, the modified hammer (weight 10 lb, fall 1.5 ft) is used to compact layers 2 in. thick and 6 in.

in diameter, 25 hammer blows per layer will represent a compactive effort of $(25 \times 10 \text{ lb} \times 1.5 \text{ ft}) \div (3 \text{ in} \times 3 \text{ in} \times 3.1416 \times 2 \text{ in}) = 6.631$ ft lb per cu in.

Note: This compactive effort is almost identical with that of the standard Proctor compaction procedure. However, under the stated conditions of the two procedures, the modified hammer produces greater density in the compacted soil than the Proctor hammer does because of the greater impact value of each modified hammer blow and because of the effect of thickness of layers. With the procedure described herein, a compactive effort of 4 ft lb per cu in. will produce approximately the same soil density as the standard Proctor procedure.

The compactive effort may be varied from a minimum value of about 4 ft lb per cu in. to as great a value as is desirable or practicable.

In selecting any compactive effort, it is important to choose the one or more compactive efforts that will produce the degree of compaction required by the conditions of the proposed use. One value of compactive effort will suffice when the range of density required by specifications will occur within a reasonable range of compaction moisture content. When this requirement is not met, or in an investigation to determine optimum compaction conditions, tests will be required at more than one compactive effort.

When the modified AASHO compaction hammer is used with specimens 6 in. in diameter and with layers 2 in. thick, the usual compactive efforts will be as follows:

1. For well graded flexible base materials use 13.26 ft lb per cu in. (50 hammer blows per layer). This group will include many soils with little or no tendency toward shrinkage or swell.
2. For moderately active soils, exhibiting some tendency toward shrinkage and swell, use 6.63 ft lb per cu in. (25 hammer blows per layer).
3. For very active soils, exhibiting high shrinkage and swell, use 4 or 5 ft lb per cu in. (15 to 20 hammer blows per layer).
4. Clean, cohesionless soils are exceptions and require individual treatment.

The compaction described above applies to materials to be placed near the surface of a soil structure. If the soil is to be placed deep in a structure where fluctuation of moisture content is limited and where the surcharge will restrain the tendency toward swell, the degree of compaction should be varied to fit the conditions.

V. Molding the Specimen

The compaction mold, with collar attached, is fastened firmly into the compaction machine (Fig. 5, text). The spacer block, with smooth face upward, is placed in the bottom of the mold. A circle of filter paper covers the spacer block.

From the mixed material prepared for one, 6- by 8-in. specimen (III, A or B), weigh out enough material to form a compacted layer 2 in. thick and place it in the mold as a loose, level layer. Maintain the grading as well as possible. Avoid placing large aggregate against the wall of the mold. Set hammer stops to provide proper fall of hammer and apply the selected compactive effort by alternately dropping the hammer and rotating the mold. Space the hammer blows as uniformly as possible over the entire surface of the layer. Scarify the surface lightly to provide bond with the succeeding layer. Readjust the hammer stops, prepare, and compact the second layer in the same manner as the first.

Remove the mold from the machine and remove the spacer block from the mold. With a small press, push the half specimen to the bottom of the mold and seat it firmly. The filter paper should remain on the bottom of the specimen. Replace the mold in the machine and compact the two final layers as before.

Remove the mold from the machine. Finish the upper surface of the specimen to a true level plane by scraping material from the high spots and compacting it in the depressions with a light tamp. The final surface is finished by tamping and smoothing with the inverted spacer block (Fig. 6, text).

Discard the filter paper, remove the collar from the mold and weigh the specimen in the mold. Record this weight with the tare weight of mold. Determine the height of the specimen by means of a dial gauge supported by a tripod which, in turn, rests on the upper edge of the mold (Fig. 7, text). The original dial setting is referred to the top of the mold. Actually, the difference in height between the top of the specimen and the top of the mold is measured. Knowing the height of the mold, the height of the specimen can be calculated (An accurate depth gauge may be substituted for this device). If the height of the specimen varies more than ⅛ in. from the intended height, the weight of material used for each specimen should be corrected.

Center a porous stone against the bottom of the specimen and, by means of a light press, eject the specimen from the mold in an upward direction. If the specimen is not to be tested in compression, it is ready to be oven dried, so that the total dry weight may be determined. If it is to be tested, place a second porous stone on top of the specimen and store the assembly overnight. Provide protection against moisture gain or loss while the compacting moisture is equalizing itself.

VI. Moisture-Density Relations

Specimes used in the determination of the moisture density curve shall not be made of remolded material. Always use new material for each specimen in order that the moisture density curve will represent the procedure to be used in molding specimens for the compression tests.

For heavy clay soils the intervals of moisture content between specimens may be as great as 2 percent; for well graded base materials the intervals may have to be as small as 0.5 percent.

With care to prevent loss of weighed materials during compaction, it usually will be possible to make a close estimate of the moisture-density curve immediately after the specimens have been molded, measured, and weighed. Thus the moisture content for the compaction of the strength test specimens may be determined and the specimens for the moisture-density curve also may be tested to determine the effect on strength produced by variation of molding moisture. The exact dry weight of each specimen will be determined by oven drying at the completion of the test.

VII. Specimens for Strength Test

The required range of initial moisture content and density of test specimens is dependent upon the conditions imposed by the intended use of the material. As indicated in Section IV, a single compactive effort, at various moisture contents, may or may not produce the range of initial conditions fixed by specifications. For each compactive effort, three sets of specimens should be prepared, each set consisting of five specimens as nearly identical as possible. One set should be compacted at or near optimum moisture content. The other two sets should be compacted 1 or 2 percent dryer and 1 or 2 percent wetter than the optimum.

Strength tests performed on the above described specimens will provide reasonably complete data over the probable compaction range. An approximation of these detailed data may be obtained by making complete tests on one set of specimens (usually compacted near optimum moisture content) and then modifying these test results in accordance with the results of uniform tests performed on a series of individual specimens representing the appropriate range of the moisture-density curve. The effect of the shortened procedure is to provide less complete information with less testing.

VIII. Dry Curing

Dry curing is applied only to specimens consisting of soils that exhibit little or no tendency toward shrinkage and cracking when dried and negligible swell upon wetting. Such soils usually have plastic indexes of 15 or less. Specimens with PI over 15 may be damaged if dry cured.

The compacted specimens, having been allowed to stand overnight, are placed in the air-drying oven (140 F) for 8 hr. Upper and lower porous stones protect the ends of the specimens. Usually, one-third to one-half of the molding moisture is removed.

Following the drying cycle, the specimens are allowed to stand overnight in the open laboratory to permit cooling and equalization of the remaining moisture. For purposes of research, the weight of each specimen may be determined at this stage.

IX. Capillary Wetting

Each specimen, with upper and lower porous stones in place, is enclosed in the pressure cell. Apply a partial vacuum to the cell, slide it over the specimen and release the vacuum (Fig. 8, text). If the specimen is of low permeability, one or two layers of slitted filter paper, long enough to overlap upper and lower porous stones, should be wrapped around the specimen before the cell is placed. The use of the filter paper wrapping will reduce the time required for capillary wetting.

Place the assembly in a water tank or pan. Use suitable spacer plates (composed of some inert substance) and adjust the free water level on the lower porous stone to a distance ⅛ in. below the bottom of the specimen (Fig. 10, text).

Apply an appropriate vertical surcharge weight to the upper porous stone. The value of this surcharge will depend upon the conditions of the proposed use of the soil. For most flexible bases and subgrade soils this value will range from 0.33 to 0.5 lb per sq in. For soils to be placed at a greater depth, the surcharge will be greater in proportion to the weight of material to be supported. The weight of the upper porous stone is considered a part of the vertical surcharge.

Next, apply an appropriate lateral pressure, in the form of air pressure, within the membrane of the cell. In most base and subgrade problems where the conditions of proposed use permit lateral swell toward side slopes or ditches, and where the depth of placement is assumed to be approximately 1 ft, the appropriate lateral pressure is estimated to be 1 lb per sq in. For other conditions of placement, the appropriate lateral pressures must be determined individually. Figure 9, text, shows the assemblies placed in water pans and attached to a constant pressure air manifold in the moist-room. If the constant air pressure device is not available, or when various confining pressures are employed, apply the required pressure to each cell individually, using an air line equipped with a valve core depressor and supplied from an auxiliary air tank at a suitable pressure. It will be necessary to measure and to adjust the cell pressure from time to time to compensate for temperature changes, leakage, etc. Specimens should be protected from evaporation or the accumulation of free water at their upper surfaces.

All specimens should be subjected to capillary wetting until absorption has practically ceased. Determine the increase in moisture from time to time by removing the assemblies from the water pans, removing surcharge weights, wiping off the excess water, and weighing the whole assemblies. It is not advisable to remove the specimens from the cells for this weighing. Very permeable specimens will reach moisture equilibrium in 2 or 3 days while specimens of very low permeability may require several weeks. It is customary to give all specimens a minimum soaking period of 10 days. Soils of low plastic index, which include most flexible base materials, will reach equilibrium within 10 days and their wetting may be considered complete without intermediate weighing. The wetting period may be shortened if justified by periodic weighing. The more plastic soils always require periodic weighing. The final wet weight of each specimen is calculated from the final gross weight by subtracting proper tare weights of cell, moist porous stones, and wet filter paper (if any).

Place the specimen on a pedestal and remove the cell, using a partial vacuum on the membrane to prevent friction. If there is a filter paper wrapping, remove it and determine its weight. Quickly measure the final height and diameter (or circumference) so that swell or shrinkage may be calculated. Avoid all unnecessary handling of wetted specimens. Replace the cell on the specimen as promptly as possible. The specimen is now ready for the compression test.

X. *Confined Compression Test*

Place the assembly (specimen with upper and lower porous stones in the pressure cell) on a loading block that is centered on the platen of the press. Place another loading block (with dial plate, if required) on the upper porous stone. Center the deformation dial on the upper block (or plate) and adjust to a convenient initial reading (See Fig. 12 and 13, text). Place the dial housing over the dial and center it on loading block (or plate). Center the spherical loading block on the dial housing as in Figure 12. An alternate arrangement, in which the top of the dial housing and the lower block of the proving ring form the spherical loading block, is shown in Figure 13. Raise the platen of the press until the loading block is in contact with the weighing mechanism but exerts no load (zero clearance). Record the deformation due to weight of blocks, housing, etc. Build up air pressure in the auxiliary air tank to—or very slightly above—the lateral pressure selected for the test. Attach the air line to the cell and apply the lateral pressure. (If testing device is similar to that in Figure 13, maintain the original deformation by raising the platen during application of lateral pressure.) Record the developed vertical load.

Apply the load maintaining a constant rate of strain. For flexible bases and subgrades the customary rate of strain is 0.15 in. per min. For soils of low permeability, proposed for use at considerable depth, the rate of strain should be adjusted to conform to the anticipated rate of loading during construction. During the test, record simultaneous readings of stress and deformation at intervals of 0.01-in. deformation. Continue the test until the load ceases to increase.

Release the load and the lateral pressure. Apply a partial vacuum to the cell and remove it from the specimen. Remove the porous stones and determine their wet weight (required as a tare weight). Determine and record the weight of stone, block, housing, etc., supported by the specimen. Air-dry the stones and determine their air-dry weight if required as a tare weight. Examine the specimen to determine shape and location of shear planes. Place the entire specimen in a weighed pan. Determine the wet weight. Dry the specimen at 110 C. Determine the dry weight of the material.

For each set of five identical specimens, use a different lateral pressure for each specimen. The range of lateral pressures depends upon the proposed use of the material. With flexible base and subgrade materials for highway construction the testing range usually is from 3 to 20 lb per sq in. For other problems a greater range of lateral pressures may be required. If pressures above 25 lb per sq in. are used, it is necessary to apply the retainer rings to the ends of the pressure cell during the test.

XI. Calculation of Data

The total vertical load on the specimen, P, at any given deformation, is the sum of the load measured by the weighing mechanism plus the dead weight of the upper stone and loading blocks, dial housing, etc. The cross sectional area of the specimen at the beginning of the test, A, is calculated from the measured diameter. From the measured height at beginning of test, h, and the observed deformation, d, at any given load calculate the percent strain, S, by the formula: $S = 100\frac{d}{h}$. The nominal vertical unit stress at a given deformation is $\frac{P}{A}$. The corrected vertical unit stress, p, taking into consideration the increase of cross sectional area during the test, is calculated from the formula $p = \left(1 - \frac{S}{100}\right)\frac{P}{A}$. From the calculated, simultaneous values of p and S, plot the stress-strain curve. (Fig. 14, text). For a given specimen, tested at a given lateral pressure, the ultimate value of p will represent the major principal stress and the applied lateral pressure will represent the minor principal stress.

From these data plot Mohr's diagram of stress (Fig. 15, text). Each individual test will be represented by one stress circle. Draw the envelope of rupture tangent to the stress circles. It is practically impossible to avoid an occasional specimen that is not identical with the other specimens in a given set. In drawing the rupture envelope, discard any stress circles that obviously are out of line. Under actual conditions of use, the soil in a structure is never unconfined; there is always some lateral pressure. Therefore, it is considered proper to define the effective value of cohesion as the intercept on the axis of shear stress formed by extending the straight line portion of the rupture envelope. The effective angle of internal friction is the measured angle between the rupture envelope and the axis of normal stress.

If the shear planes in the test specimen pass through the upper or lower faces of the specimen, or if the shear planes pass through the sides of the specimen near the upper and lower faces and have horizontal "shoulders" in the body of the specimen, then the specimen is not tall enough for the given diameter. As a result, the indicated strength of the specimen is too great by an unknown amount.

The dry density of each specimen as molded can be calculated from the dry weight of the specimen and its original measured volume. The moisture content as molded can be calculated from the original, net wet weight and the final dry weight.

Supplementary data that may be of use in construction or research can be calculated from the recorded weights and measurements. The moisture content of the specimen as dry-cured can be determined from the net dry-cured weight (gross weight minus the weight of the two air-dry porous stones) and the final dry weight. The moisture content after capillary wetting can be determined from the net weight after wetting (gross weight minus weight of cell minus weight of the two moist, porous stones) and the final dry weight. A rough check on this value can be calculated from the moisture determination after the specimen has been tested. The percentages of vertical, diametric, and volumetric swell can be calculated from the measurements made before and after capillary wetting. The dry density after capillary wetting can be calculated from the final measured volume and the final dry weight. Supplementary determinations of the bulk and specific gravities of the component materials, together with the recorded data, will make possible a volumetric analysis of the structure of the specimens at all stages.

DISCUSSION

W. H. Campen, *Omaha Testing Laboratories:* Mr. McDowell's paper is a valuable contribution to the field of soil stabilization. Among other things he suggests a number of modifications to the standard method of determining the moisture-density relationship of soils. In addition he calls attention to two important phenomena. One deals with the possible effects of overdensification and the other with the effects of drying and wetting on strength.

I wish to comment on these points and will take them in the order named. The suggestions for the density test pertain to the use of fresh samples for each trial, the use of the entire sample, the use of a larger mold, and the use of a variable compactive effort. These suggestions are sound as will be attested to by many technicians and engineers. I call attention to the fact that some of these suggestions were recommended in a sub-committee report last year.[1]

It seems to me however that Mr. McDowell's method of determining the desirable compactive effort and density is not very scientific as it is based on personal judgment. Furthermore the method has the effect of using compactive effort and densities as the final answer whereas they should be used as a measure to an end. In this type of research we all have the final strength in mind. Therefore strength is the governing factor. I am of the opinion that eventually strength standards will be set up. After that in evaluating a soil mixture its density corresponding to the desired strength will be determined. Last year we proposed a method for making these determinations[2].

As far as the over-densification phenomenon is concerned I do not wish to doubt the results but I am reluctant to believe them. I suggest that this type of test be made on more soils and with other available methods before its effects are considered in design. Personally I hope the indication will not be confirmed for the reason that its application will limit to low levels the strength values which may be developed in certain types of soils. I might add that our own limited information on the subject does not confirm Mr. McDowell's findings.

Considering the effects of drying and wetting on strength, the data in Table 1 for soil-aggregate mixtures give positive indications that the treatment increases the strength. This effect may be accounted for by two lines of reasoning. First, on drying, the samples may shrink and thereby become denser. On subsequent wetting they may not regain their original size even though they may take up as much water as they contained at the time of casting. Second, the distribution of moisture may not be uniform, the chances being that the top of the specimen, where the strength test is made, is not as wet as the bottom. Either of these conditions would give higher strength results.

[1] Progress Report of Subcommittee on Methods of Subgrade, Sub-base, and Base Preparation for Strength, W. H. Campen, Chairman, *Proceedings*, Highway Research Board, Vol. 25, (1945).

[2] W. H. Campen and J. R. Smith, "Bearing Index as a Criterion for the Maximum Density Requirement," *Proceedings*, Highway Research Board, Vol. 25, (1945).

The data in Table 2 for fine grained soils are not very convincing. It is rather difficult to analyze the strength results because the specimen as compacted usually contained high air voids as shown by the absorptions. Generally speaking however the strengths after wetting with and without previous drying are proportional to the moisture content. If any strength is developed by the drying process it is slight and spotty.

In connection with effects of strength on drying I doubt if advantage can be taken of the possible beneficial effects. The cost involved in the field would probably more than offset the gains and it would be practically impossible to control the process under all ground and weather conditions.

MR. H. P. CAROTHERS, *Texas Highway Department:* It is noted from the discussion by Mr. W. H. Campen that there is considerable reluctance to believe the over-densification phenomenon of swelling clays and he suggests that more tests be made before its effects are considered in design.

We agree that more tests should be made, but wish to bring out that the effects are recognized and are being used in design.

For years while heavy compaction on the dry side has been preached, many of our resident engineers, from experience, have been warning against such over-densification on some clays and indicating the belief that some soils will seek some "natural condition of density and moisture" under the pressure of pavement and traffic load.

Our first move has been to locate the extremely high swell-low strength soils that are causing major "heaves" in pavements and extensive pavement failures. These are marine deposited marls and are easily located from geologic maps and inspection of pavements constructed over these formations.[1] A survey of several hundred miles of pavements over this formation shows that regardless of type of pavement, amount of compaction, method of construction, or type of subgrade treatment, serious swell has been experienced and thin pavements (both rigid and flexible) have serious failures. Incidentally, a geologic map of the United States indicates that Texas probably has as much or more of this type of deposit than all the rest of the States combined.

Figure 2, "Swell and Strength Test Results" of the paper shows results of tests which were made on the black clay derived from the yellow marl (joint clay) of the Taylor Marl formation. The strength tests were made by a small punching test after capillarity. The Texas Highway Laboratory has recently completed confined compression tests (run after capillarity) that verify the over-densification phenomenon and show that the parent Taylor Marl is weaker than the black clay derived from the parent marl. Also, under similar conditions the parent marl swells about 1.5 times that of the black clay topsoil. This verifies the observation from survey of pavements on this formation, which indicated that the closer the marl was to the pavement (less than about 5 ft) the greater the waving and breakup of the pavement.

Also, we have noted in a report "The California Bearing Ratio Test as applied to the Design of Flexible Pavement for Airports" made by the U.S. Waterways Experiment Station, Vicksburg, Miss., that in one instance of a high-swell clay the soaked CBR strength for the standard proctor compaction was greater than for the heavier modified proctor compaction.

In addition, reference is made to the "Report of Committee on Warping of Concrete Pavements," in the 1945 Proceedings of the Highway Research Board. It will be noted on Page 243 that experience and laboratory tests have demonstrated that high swelling soils have a definite moisture content and compaction which will result in what is termed a "no swell-no shrink condition."

For use in flexible pavement design and to help settle the question of what moisture and density to construct high swelling or highly compressible subgrades and for molding samples for tests with the compression test, this writer wishes to suggest the following procedure for further research:

1. Mold samples at densities and moisture contents both below and above those anticipated to be secured in the field.
2. Run consolidation test on the samples (subjecting to capillarity) at unit pressures of say 2, 4, 8, 16, 32, and 64 lb per sq in. pressure. Also, run

[1] Reference is made to a report on "Relation of Geology to Road Design—Taylor Marl," Texas Highway Department.

rebound curves and reconsolidation curves.

3. Measure for any swell or consolidation at each load, and determine density and moisture at each load. Swell under load would indicate less compaction or compaction with greater moisture content or lower air voids.
4. Estimate weight of pavement and traffic load (curves may be prepared from theory of stresses under circular loads).
5. Consolidation under traffic load would indicate greater compaction needed. Much increase in moisture content under traffic load would indicate need for compaction at higher moisture content or heavier compaction.
6. From the above it may be possible to estimate the proper density and moisture at which to test and construct subgrades.

Concerning details of the strength test reported, the following comments are offered:

1. It is noted that the maximum size of aggregate and depth of compacted layer are the same, namely, 2 in. It appears that when the aggregate is over 1 in., the layer should be increased to about twice the size of maximum aggregate.
2. The size of specimen used is 6 in. by 8 in. It is suggested that a height of two times the diameter be used until it is definitely shown that shorter specimens may be used. It appears that the diameter of the specimen should be at least four times the maximum size aggregate. Also, specimens of 4-in. diameter and 8-in. height have been successfully used on soils passing the 40-mesh sieve, and is a convenient size when converting available laboratory equipment for use in the compression test.
3. It is suggested that a soaking load approximating the estimated traffic load plus weight of pavement be used in the test (also, greater lateral support), since it is believed that this will be necessary to secure a good correlation of theoretical analysis of estimated stress and measured strength with actual service records of pavements. This correlation can best be made by measuring the strength of typical soils of a given geological formation, estimating pavement design by theory and then checking against pavements constructed on the same formation Further checks should also be made on undisturbed samples taken from the existing pavements and subgrade. When undisturbed samples cannot be taken, they should be remolded to existing density and moisture content. Extreme caution should be exercised when checking flexible pavements less than about 10 years old and rigid pavements less than about 15 or more years, since in many cases it has taken this time or more to prove adequate design.
4. Caution should be exercised in allowing the samples to remain in an unconfined state for any period of time.
5. The extreme time of soaking and tests at five lateral pressures are satisfactory for research but should be reduced as much as possible for practical application.
6. The blanket factor of safety with relation to ultimate strength suggested for application should be used with caution, since the stress-strain characteristics of various soils vary greatly. Design must be made to prevent excessive deflection as well as over-stress.

The use of these strength tests in combination with theoretical stresses and estimate of pavement design checked against existing pavements on the same geological formation is rapidly removing much confusion caused by attempted application of design from soil constants alone. It appears that the logical approach to flexible pavement design is as follows:

(1) Determine traffic load (2) Estimate stresses in pavement caused by the load (from the elastic theory) (3) Measure strength of the materials (with compression test) (4) Design pavement to prevent over-stress and excessive deflection (5) Before use, check this design against existing pavements on the same or similar geologic formations.

Mr. McDowell, *Closure:* Mr. Campen's discussion brings up some points which should be clarified:

1. The need for a more scientific and less arbitrary method for determining the density desired during construction than was proposed in the paper. He suggests the possibility of basing this upon strength.

Until more information is available on the relation of densities to strengths of soils it is more arbitrary to select densities for low to non-swelling soils on the basis of strength tests than it is to select densities on the basis of field compaction experience. This is because increased densities are accompanied by increased strengths, thereby giving no particular indication of the highest practicable density for use. In the case of swelling soils it is possible to use strength and swell tests (as long as specimens during absorption are fairly free from restraint) to assist in the selection of the density desired. Our approach to this problem may not appear very scientific but it appears to be the best available at present.

2. He is reluctant to believe that the swelling effects of clays are significant insofar as design is concerned and doubts if additional tests on other clays will substantiate the findings.

Any one who believes that clay soils can not swell sufficiently to cause rough riding pavements will be convinced otherwise by examining the profile conditions of various heaves at locations throughout the clay areas from which the clay soils were selected and tested as shown in Figures 1 and 2. It is agreed that additional tests should be run on various types of clay soils. Additional swell tests using thin specimens in 1-in. height consolidometer rings and 6-in. diameter by 8-in. height specimens in flexible membranes for soil 39-11-MR have confirmed the results shown in Figure 2 except that the percentages of swell indicated by the additional tests were approximately two greater than those shown in the report. Perhaps the lower percentages of swell obtained by the original tests were due to the presence of some restraint being furnished by side wall friction of the 2½-in. height rings. The confined compression tests also showed the same over densification phenomenon for upper layers of subgrade consisting of this soil. Similar swell tests on the parent marl for this soil when similarly compacted gave percentages of volumetric swell in the order of 1½ times as great as those obtained for soil 39-11-MR. It is believed that there are other Texas soils of even greater swelling characteristics than have been mentioned. This should not be too astounding if it is recognized that many Texas soils are impregnated with various amounts of bentonitic types of materials. In some localities it is possible to find deposits of practically pure bentonite.

The swell tests shown in Figures 1 and 2 appear to be approximately correct; however, it was not the intent of the report to infer that all clay soils will swell excessively. It is commonly known that certain types of clays, such as Kaolin, etc. will produce relatively small amounts of swell. Clays of different geological origins may vary considerably in swelling characteristics. It should be noted that it is proposed in the test procedure of the report to measure the swelling characteristics. Therefore, if a low to non-swelling clay soil is being proposed for use in construction it will be detected by the tests and should not be treated in the same manner as is proposed for medium to high swelling clays.

3. He questions the moisture distribution of the crushed stone specimens referred to in Table 1 and suggests that their strengths may have been affected by testing the "top or dry end" of the specimen.

These specimens were tested in compression after capillary absorption and equalization of moisture films to obtain the data shown in Table 1 rather than by penetrating the end of the specimen as is inferred by Mr. Campen. Since these specimens had ceased to absorb additional water and they bulged at their midpoints during compression there is justification in our belief that the moisture contents in these specimens was fairly uniform throughout the height of the specimens. Therefore, the question of non-uniform distribution of moisture does not satisfactorily explain the increased strengths due to dry curing. Increased densities due to dry curing may partially account for the phenomenon.

Under certain conditions, strength increases resulting from dry curing may also be partially accounted for by the principles of physical chemistry relating to colloids. Some soils contain lyophobic (in this case hydrophobic) colloids whose hydration depends upon an electrical double layer which is readily discharged by drying. Rehydration is very difficult. Glasstone in his "Textbook of Physical Chemistry" describes lyophobes (here, hydrophobes) as sols that are not reversible (from set gels to sols) except under the original conditions of formation. Another type of clay particle known as the lyophilic (or hydrophilic) type of colloid is readily rehydrated after drying. However, the colloidal theory alone is not sufficient to account for all of the phenomena observable under various conditions. Obviously, other physical and chemical principles are involved.

results for dried and nondried specimens for this silty soil 39-10-MR, as shown in Table 2 is somewhat spotty. This is consistent with the statement in the report that the stength of some soils may not be benefitted by dry curing.

5. Due to economy, he believes it impractical to take advantage of the benefits of the dry curing factor and that it would be impossible to control the process under all ground and weather conditions.

The practical aspects of the dry curing process as proposed in the report may not be apparent if the method is viewed strictly

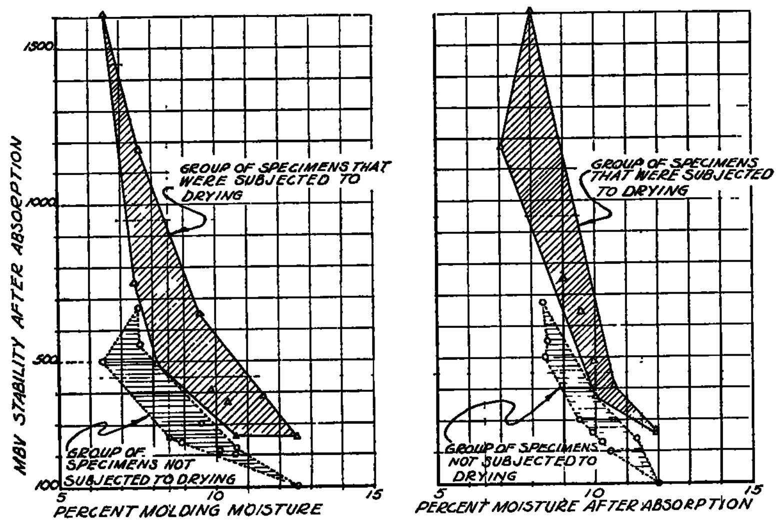

Figure A. Relation of Strength to Moisture Content—Soil 39-7-MR

4. He states that the data in Table 2 are not very convincing in that final strengths are proportional to moisture content and that any effects of dry curing are slight and spotty.

The strengths after capillarity shown in Table 2 for soil 39-7-MR may be roughly proportional to moisture content as is shown in Figure A; however, a close examination of the data shows a great deal of difference between the final strengths of specimens that were dried as compared to those that were not dried. It may be noted that for any given percentage of molding or absorbed moisture the strength of the dry cured specimens of this soil is always the greatest. It is agreed that the trend of strength test as one that must be carried out immediately in a given length of time as is practiced in the laboratory. Actually in the field, the drying of individual layers of compacted low swelling soils may be carried out satisfactorily over a considerable length of time in which even some weather fluctuations may occur. Most construction operations are usually such that the time for drying out approximately 50 percent of this compaction moisture, as is proposed in the report, does not delay the contractor except in exceptional cases. Ordinarily this process improves the contractor's working conditions. Therefore, the process appears to be an economical one that is naturally followed in the field as a matter of necessity. Ordinarily it would be uneconomical to prevent this process from taking place because, to do so, blanketing of the layers to prevent evaporation would be required. This is attested to by the fact that some engineers are skeptical of the economy of retaining the compaction moisture content in swelling clays as is proposed in the report. It is unfortunate that the dry curing process has been overlooked to date in most laboratory investigations of granular soils and flexible base materials, because so many of these test results appear to have been affected more greatly by compaction moisture than by absorbed moisture.

Mr. Campen's discussion has done much to point out various phases of this report which need further clarification.

COMPACTION AND STRENGTH CHARACTERISTICS OF SOIL[1] AGGREGATE MIXTURES

By E. J. Yoder and K. B. Woods

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SYNOPSIS

This paper reports one of the researches conducted by the Highwaz Research laboratories of Purdue University co-operating with the State Highway Commission of Indiana. A series of compaction and strength tests were performed on four soil-aggregate mixtures to determine their compaction and wet strength characteristics under varying soil contents.

On the basis of the test data, it was indicated that, for a given gradation of an aggregate, there is an optimum soil content at which maximum densities are attained. The optimum soil content where strengths are concerned is somewhat less than that indicated by the compaction tests. Maximum densities do not necessarily mean maximum strengths whenever soil-aggregate mixtures containing varying percentages of soil near the optimum are compared. The tests on mixtures of soil and crushed stone resulted in the highest density and strength values of the materials, with the soil-gravel, soil-sand, and soil-dune sand mixtures resulting in the next highest values in the order given.

It was concluded that insofar as densities and strengths are concerned, a small quantity of soil mixed with granular materials is desirable, but that larger quantities are detrimental.

In recent years performance surveys of various types of pavements have been made to determine the causes of numerous failures observed each spring in both rigid and flexible pavements. It has been observed that, in most cases, there is a distinct difference between the performance of pavements located on plastic, silty-clay soils with no base courses and those constructed on granular or semi-granular materials. The importance and need for increased use of granular base courses has been shown in connection with rutting of flexible pavements and traffic-bound roads. Likewise, rigid pavements constructed directly on plastic soils have been found to pump badly, particularly under conditions of heavy traffic.

One of the primary problems, wherever granular and semi-granular materials are used, concerns the amount of fine material that should be permitted in an aggregate, and the effect of the fine material on the compaction and wet-strength characteristics of soil-aggregate mixtures. Fine-grained soils have low unit weights, good supporting power when dry, have high capillarity, have poor drainage characteristics, and have very low supporting power when they contain excessive water. On the other hand, granular materials have high unit weights, are not

[1] The word soil as used in this text refers to material that will pass a No. 200 mesh sieve.