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Bulletin 93

Soil Density and Stability

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2101 Constitution Avenue Washington 25, D. C.

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Soil Density and Stability

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Contents

Selection of Densities for Subgrades and Flexible-Base Materials

CHESTER McDOWELL, Senior Soils Engineer
Texas Highway Department

● THE need for a realistic method of selecting, from tests, densities which are comparable to ideal field conditions has led to the use of the term "compaction ratio." It expresses that portion of the possible densification that it is desirable to accomplish in changing a material from a loose to a very-dense state. It has simplified matters materially by making it possible to select or predetermine desirable roadbed densities for construction involving a wide range of soils and base materials by means of unit-weight and moisture-density tests.

Our experience with the compaction of soils involving all gradations from clays to granular base materials has indicated the need for a means of expression for significant states of grain structures which affect relative supporting values. Investigations of expansive properties of clays as related to strength and compaction, together with experience in molding specimens consisting of large particles, often have shown the need for such an expression. Data relative to this subject were presented at the annual meeting of the Highway Research Board in 1946. Our procedure (see THD-83 in the appendix) for compaction of samples consisting of large, top-size aggregates in large molds has done much to help establish a correlation between laboratory and field compaction. Additional investigations, including in-place field densities, have shown that it is not practical to obtain such an expression by use of percent compaction or percent density and that the use of percent compaction as a means of field control may cause overcompaction of clays and undercompaction of granular soils. Occasionally, harsh mixtures of base materials are encountered which cannot possibly be compacted to a density as high as that calculated for the total material from density tests of the minus-¼-inch or No. 4 material, specific gravity of the retained portion, etc. Rolling of most soils which are not of a harsh nature produces the required percent density or compaction with so little compactive effort, and the material is so loose after rolling that our field forces often do not consider such compaction as satisfactory. Hence, there has been a disturbing trend toward relying solely upon visual inspection in lieu of tests to control compaction. In fact, in many instances the rigorous use of specifications certainly would have produced compaction inferior to that obtained by visual inspection. This trend will continue until better methods of testing and means of expressing the selection of densities are employed.

Donald M. Burmister indicated concern over these same matters in his excellent article: "The Importance and Practical Use of Relative Density in Soil Mechanics," published in Volume 48, Proceedings ASTM, where he described the problem in the following statements:

> the commonly used term, percent compaction, expressed as a simple percentage of the maximum density only, is not significant but is actually misleading, because it is based upon the obvious fallacy that zero percent compaction can represent a unit weight of zero for any soil, whereas there is always a definite physical limit to the minimum density or loosest state. . . . A field control requirement of a certain percent compaction gives a false sense of a good degree of compaction, which does not in fact exist. To the average engineer or contractor 100 percent compaction represents perfection and 90 percent compaction, according to common standards of engineering practice, is considered to be very good indeed. . . . It is evident that field-compaction requirements and specifications based on percentage compaction are not good enough for the construction and treatment of subgrades and of high-quality base courses for airport-runway pavements intended to support heavy airplane wheel loadings, particularly under the vibrational effects. Such requirements are inadequate because they cannot properly fix the lower density limit considered permissible and acceptable in earthwork construction, which will minimize objectionable consolidation of granular soils and objectionable lateral plastic displacements of clay-soils under such loadings. Only 100 percent compaction has any real practical significance. The use of the term, percent compaction, in specifications should therefore be discontinued.

In attempting to work out a means of using relative density for analyzing this problem, a similar but different expression was discovered which simplified matters: compaction ratio is defined as the ratio of the difference between actual and loose densities to the difference between dense and loose densities, expressed as a percentage and is represented by the expression $\left(\frac{D_A - D_L}{D_D - D_L}\right)100$. It expresses the degree to which it is desirable to compact soil materials on the basis of the compactibility of the matêrial. The experienced technician often can select a density to which it is desirable to compact a material by investigating the effects produced by a variety of compactive efforts, as suggested in THD-83 in the appendix. By reference to the procedure involving the use of compaction ratio, THD-110, it is possible for the inexperienced technician to select proper densities on the basis of two specific tests, i. e., a unit-weight determination and a moisture-density test. The particular densities referred to above are obtained by the use of the following techniques:

1. Loose density D_L is obtained for clean sands and granular materials by running rodded unit weight tests (ASTM Designation C 29-42) and is expressed in pounds per cubic foot dry. For other soils containing small amounts retained on the No. 40, D_L is obtained by determining the dry unit weight per cubic foot of the soil shrinkage pat which has been molded at the liquid limit condition and corrected for any plus-No. -40 material which under field conditions will be floating in the soil. This loose structural condition perhaps is analogous to a formation freshly deposited by water.

2. Actual density D_A is the dry weight per cubic foot which experience has proven to be desirable and practical. Table B of THD-110 shows that a close approximation of this density can be obtained in the laboratory by employing a particular compactive effort the intensity of which depends upon the character of the soil. D_A also may be obtained by calculation, as will be shown later.

3. The term dense density "D_D" is obtained from the peak of an optimum moisture curve run at a high compactive effort of approximately 30 ft. -lb. per cu. in.

From the data in Table A it may be shown that the percent compaction desired, (D_A/D_D) 100, varies from 87 to 99 percent for all soils tested, and that the dry-rodded unit weight for some of the soils, when expressed as a percentage of D_D, varies from 84 to 92 percent; therefore, by specifying 90 percent density, the resulting density would not be above the dry-rodded unit weight in some cases. Hence, a more-rational approach was sought by use of compaction ratio which is expressed as follows:

CR in percent = $\left(\frac{D_A - D_L}{D_D - D_L}\right)100$. Figure C and Table A of THD-110 show that a good correlation exists between compaction ratio and D_L for widely varying types of soils. The relation may be expressed by the equation $CR = 58 + \frac{D_L}{3.9}$. Then, actual density $D_A = \frac{CR}{100}(D_D - D_L) + D_L$. Since CR can be expressed in terms of D_L, it is obvious that we have eliminated one variable and it is possible to determine the desired D_A in terms of D_L and D_D. Figure 1 shows how simple it is to use this method.

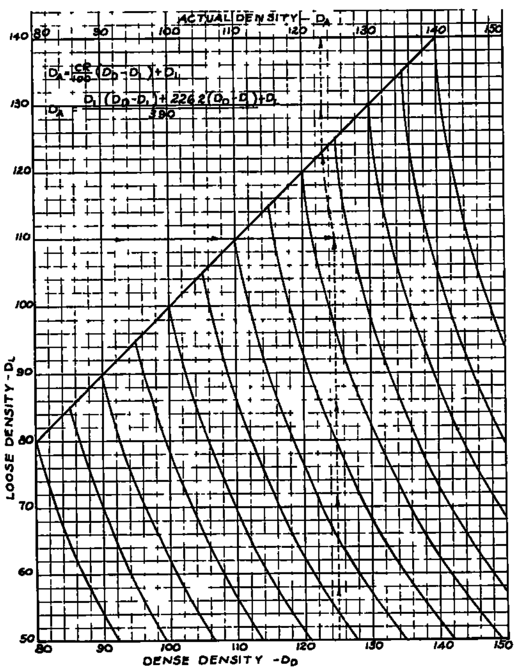


Figure 1. Chart showing relations between D_L, D_D and D_A.

CONCLUSIONS

It is believed that desirable densities can be predetermined successfully by using a compaction ratio ranging between 70 and 90, depending upon the D_L characteristics of the material. It should be emphasized that the densities selected by the use of simplified tests in conjunction with compaction ratio are comparable to those obtained by running extensive volume-change and strength tests. It is also important that moisture contents of high-volume-change soils be controlled during and following construction in order to obtain the greatest subsequent strength, small amounts of swell, and a minimum of dry weather shrinkage cracks. In order to do this it is necessary to select ideal initial moisture-density relations for construction in such a manner as to avoid volume change in amounts sufficient to cause excessive movements and loss of soil stability. It is also necessary that such selected soil-water systems be protected from excessive evaporation so as to reduce destructive, dry weather cracking. Methods of control of clays proposed in THD-110 offer many possibilities and as evidence of their practical use, it is pointed out that the methods proposed are based upon field performance such as that of the Houston Expressway. The clay subgrade soil on this project was sealed with asphalt membranes, and moisture-density relations have remained practically unchanged for over 5 years, although dry weather cracks occurred extensively in shoulders overlying slope membranes.

It is also concluded that density selected by the proposed procedure is a desirable state for subgrade and base construction, including expansive clays, and can be obtained by the application of normal amounts of rolling; however, no attempt is made to present details of field control for such rolling in this paper. Nevertheless, a few remarks on this subject seem appropriate at this time. In the first place, everyone seems to need better equipment and techniques for measuring in-place densities. We hope to overcome these difficulties soon. In the second place, the condition of the soil as determined from tests and use of compaction ratio should be specified in lieu of trying to accomplish the same end by specifying hours of rolling. It is believed that better control will be obtained through testing, and once contractors are aware of the moisture-density conditions desired, they will find the best and most-economical ways of obtaining them. The amount of rolling required to obtain minimum density requirements may not be adequate for finishing purposes; therefore, the amounts of rolling hours for finishing probably should be continued and handled as a separate item.

ACKNOWLEDGEMENTS

The writer is indebted to many who have contributed, encouraged or assisted in the development of compaction tests. A few of these people or organizations have been mentioned in the text; however, the writer feels that the contributions of the members of the Soils Section and other members of the Materials and Tests Division of the Texas Highway Department have been a major factor in making this report possible.

Appendix

PREPARATION OF SOIL AND FLEXIBLE BASE MATERIAL

TEXAS HIGHWAY DEPARTMENT

THD-53
Revised 1953

Foreword

This method describes in Part I the preparation by washing of soil samples for mechanical analysis and the determination of the soil characteristics. This method is in close agreement with AASHO Designation T 146-49 but differs from Standard Dry Preparation Methods AASHO T 87-49 and ASTM D421-39 for materials which contain particles larger than the 40 mesh. Part II describes the preparation of materials for tests (compaction, triaxial, and stabilization) which require laboratory compacted specimens.

PART I: WET PREPARATION OF DISTURBED SOIL SAMPLES FOR SOIL CONSTANTS AND HYDROMETER ANALYSIS TESTS

Prepared Soil Binder

The purpose of the "preparation" is to separate the material into sufficiently small sizes so that all of the material will be mixed thoroughly and uniformly wetted in the making of the various subsequent soil tests. That portion of the material passing the Standard U.S. 40-mesh sieve shall be known as "Soil Binder". It shall be separated from any larger particles without fracture of the individual grains. A representative sample shall be selected according to the method outlined in THD-50.

Apparatus

The apparatus consists of: (1) scale of 30-lb. capacity, sensitive to 1/100 lb; (2) set of standard screens and sieves for screening small samples and another larger set, preferably with shaker, for screening large samples; (3) 6-inch mortar; (4) stone pestle; (5) rubber-covered pestle; (6) scoop; (7) air-dryer with temperature range of 120 F. to 160 F.; (8) supply of pans; and (9) small siphon tube.

Size of Sample

For samples predominantly soil, approximately one quart or three pounds will be required. Larger samples should be selected depending on the amount of aggregate. For materials containing aggregate, the sample should be large enough to produce one quart of soil binder (minus No. 40 mesh) or approximately 6 to 12 lb. total material. Larger samples are always satisfactory. Such materials should have all aggregate crushed to the maximum size permitted in its contemplated use, and care should be taken to prevent loss of the fine material produced by the crushing process.

Preparation of Test Sample

1. Dry the soil sample in an air drier or oven of a temperature not to exceed 140 F. Higher temperatures than this may change the characteristics of the soil binder.

2. Determine whether the material has any particles larger than 40-mesh in size. This can be done by visual inspection or by slaking a small representative portion in water. If the material contains no particles larger than 40 mesh in size, or if the amount of particles larger than 40 mesh is small and can be easily distinguished by eye, these particles should be removed by hand and steps 9 and 10 followed in the preparation of the soil binder.

3. For materials containing an appreciable amount of aggregate, the air-dried sample selected for the purpose of mechanical analysis and physical tests should be weighed and the weight recorded (uncorrected) for hygroscopic moisture. The sample shall then be separated into two portions by means of a No. 40 mesh sieve. The material passing the No. 40 sieve shall be identified and set aside for later recombination with additional binder as described in steps 7 through 10.

4. The material retained on the 40-mesh sieve shall be placed in a milk pan, covered with clear water and allowed to soak for a period of 2 to 24 hours. If the sample is immersed for only a few hours, care shall be taken to see that no lumps containing soil binder remain in the aggregate. The slaking time for base materials can be determined as described in THD-54.

5. After slaking, the material shall be washed on a No. 40 sieve in the following manner: The empty sieve shall be set in the bottom of a milk pan and the liquid from the wet sample poured into it. Enough additional water shall be added to bring the level of the water in the pan approximately ½-inch above the mesh of the sieve. A small portion of the soaked material shall be placed in the water on the sieve and stirred by hand at the same time the sieve is agitated up and down. If the material retained on the 40 sieve contains lumps that have not disintegrated, but which can be crumbled or mashed between the thumb and finger so as to pass the 40-mesh sieve, such lumps shall be broken and washed through the sieve into the pan. After all of the soil binder appears to have passed through the sieve, the sieve then shall be held above the pan and the retained material shall be washed by pouring a small amount of clean water over it and letting the water drain into the pan. The washed material retained on the No. 40 sieve shall be transferred to a clean pan.

6. The procedure of step 5 shall be repeated until all of the soaked sample has been washed.

7. The material retained on the 40 sieve shall be dried and dry sieved on the No. 40. The portion passing the 40 sieve in this operation shall be added to the material passing the 40 sieve obtained in step 3. The retained material shall be weighed and then set aside for use in the screen analysis of the coarse material.

8. After all of the soaked material has been washed, the pan containing the fine soil and wash water shall be set aside and not disturbed until all of the soil particles have settled to the bottom of the pan and the water above the soil is clear. As much of the clear water as possible shall then be decanted or siphoned off. In some instances, the wash water will not become clear in a reasonable length of time, in which case, the entire water contents must be evaporated. The soil remaining in the pan shall be dried at a temperature not to exceed 140 F.

9. The dried soil binder shall be pulverized to pass the No. 40 mesh sieve by suitable hand or mechanical methods, taking care not to fracture the original grain sizes and then be combined with the material passing No. 40 sieve obtained by the procedure described in steps 3 and 7. After all the soil binder has been prepared to pass the No. 40 sieve, it should be weighed and thoroughly mixed to produce a uniform homogeneous mixture of all particles.

10. Add the weight obtained in steps 7 and 9 for the total weight of sample. This weight is used as a check against the original weight and to calculate the percent of soil binder in the material.

$$\%\text{ soil binder} = \frac{\text{Wt. passing No. 40 sieve}}{\text{Wt. of total sample}} \times 100.$$

The soil binder passing the No. 40 mesh sieve obtained as described in paragraphs 2 and 9 shall be used in performing the hydrometer analysis and for determination of the soil characteristics. Materials which contain aggregate prepared by this method usually have a higher percentage of soil binder (minus No. 40 mesh) and show more plasticity than those prepared by Standard Dry Methods ASTM D 421-39 and AASHO T 87-49.

PART II: PREPARATION OF SAMPLE FOR COMPACTION, TRIAXIAL AND STABILIZATION TESTS

1. Select approximately 200 pounds of material to prepare. Samples should be obtained "crusher run" or as near the condition in which they are to be used on the road as possible. Caliche, rock and similar materials obtained from test holes should be crushed in laboratory to pass 2" screen. Spread sample on clean floor to air dry. Do not slake or oven dry any of the material.

2. After the sample has air dried sufficiently to handle, the necessary steps to be followed will be governed by the type of material encountered: (a) Clay and sandy clay subgrade materials having no appreciable amount of aggregate present shall be crushed to pass 1/4" screen. The sample is then separated into two portions by means of the No. 20 mesh sieve. This provides a material that can be handled without segregation; (b) clay subgrades containing aggregate should be prepared by breaking up all of the clay lumps to pass a 1/4" screen without breaking up the aggregate. This may be done by means of a plastic mallet, rubber-covered tamp or similar hand tool. The material shall then be screened as described in (c). (c) The desired quantities of caliches, crushed rock, gravel and sand shall be screened over the following suggested sizes: 2", 1 1/2", 1", 3/4", 1/2", 1/4" and No. 10 mesh.

3. Weight the various pans full of sizes including the portion passing the No. 10 mesh and compute the percentages. These percentages are not to be used as a true screen analysis, but are to be used in recombining the sample for individual specimens.

4. Determine the hygroscopic moisture content of the total sample as follows: Weigh a representative portion of minus No. 10 mesh material that has been thoroughly mixed. Oven dry this sample at 230 F. and weigh. Compute the percent hygroscopic moisture based on total material.

$$\%\text{ Hygro. Moisture} = \frac{\text{Wet Wt.} - \text{Dry Wt.}}{\text{Dry Wt.}} \times (\%\text{ minus No. 10})$$

In this determination, it is assumed that the moisture in the plus 10-mesh material is negligible.

GENERAL LABORATORY COMPACTION TEST FOR MOISTURE-DENSITY RELATIONS FOR SOILS

TEXAS HIGHWAY DEPARTMENT

THD-83

Scope

This method of test is intended for determining the relationship between the moisture content of soils and resulting densities (oven-dry weight per cubic foot) when the soil or flexible base material is compacted in the laboratory as specified herein. This procedure differs from the Standard ASTM D 698-42T, AASHO T 99-49 and THD-84 methods in that the test pertains to the use of the entire sample consisting of large sizes, the use of a larger mold, the use of a variable compactive effort, and the use of a fresh sample for each trial.

Apparatus

(1) Compactive device (see Figure A) with base plate to hold 6 inch I. D. molds equipped with 10-lb. ram and adjustable height of fall. Striking face of 10-lb. ram-40-deg. segment of 3-inch radius circle; (2) compaction mold with removable collar — 6-inch inside diameter and 8-inch height; (3) measuring device, micrometer dial assembly (Fig. B) for determining height of specimen; (4) scale, rated 30 pound capacity, sensitive to 1/100 lb. ; (5) extra base plate to hold mold (Fig. C); (6) press, low capacity, to eject

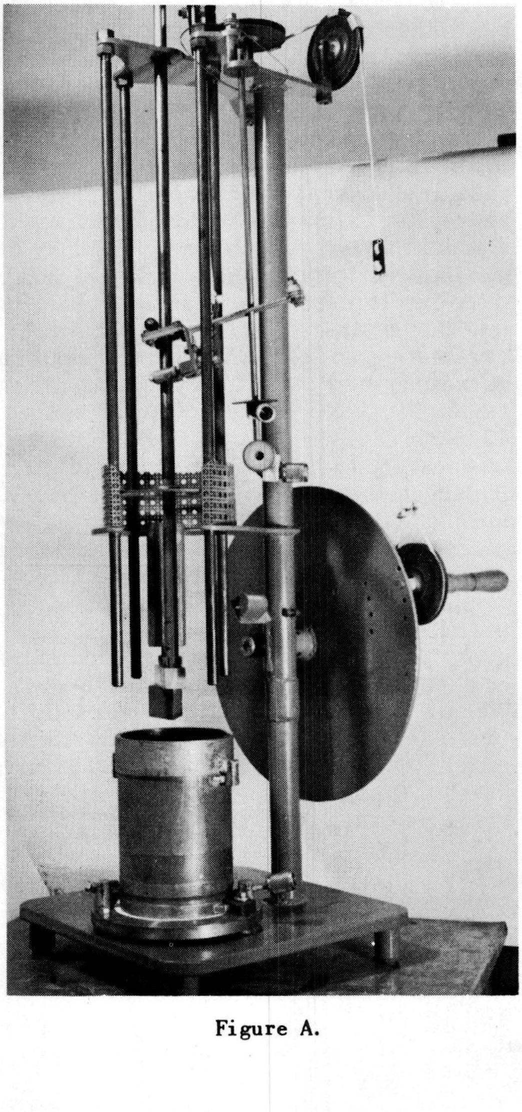


Figure A.

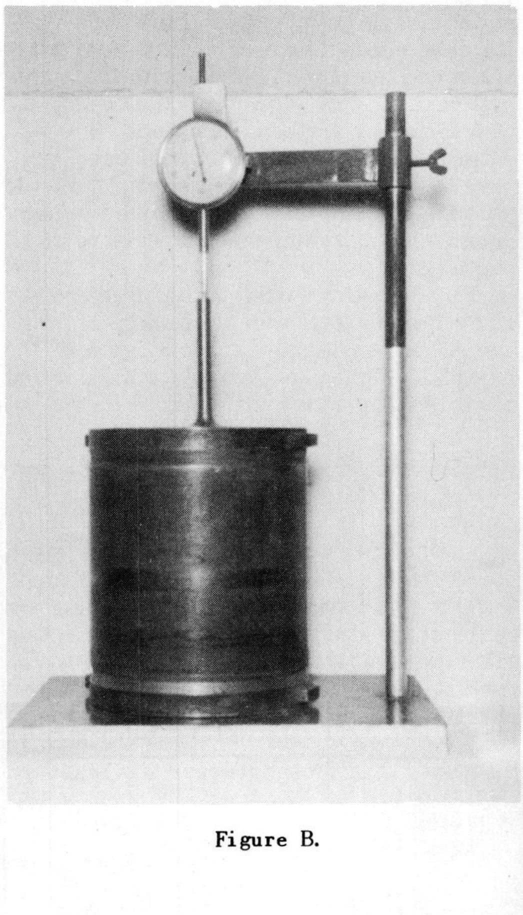


Figure B.

specimens from mold; (7) drying oven, controlled to 230 F.; (8) pans, wide and shallow for mixing materials; (9) a supply of small tools, hammer, plastic mallet, level and others; and (10) circular porous stones slightly less than 6 in. in diameter and 2 in. high.

Calibration of Mold

The method of calibrating a 6-inch mold is different from that employed for regular Proctor and similar molds. In the case of the 6 inch mold, the total volume of the mold would be of no use since the mold is neither filled nor trimmed off. Instead, a volume of per inch of height is employed. The diameter of the mold is measured (by means of a micrometer caliper and a dial micrometer) at the ends and several intermediate points to obtain an average diameter. The average diameter is used to calculate a mean cross sectional area of the mold. The area thus computed is then used to compute the volume in cubic feet for one inch of height. Thereafter, the volume for any height can be found by multiplying the above volume by the height of the specimen in inches.

$$\text{Vol. per inch} = \frac{\text{area in sq. in.} \times 1 \text{ in.}}{1728} \text{ cubic feet}$$

Vol. of specimen = volume per lin. in. x height of specimen in inches

Preparation of Sample

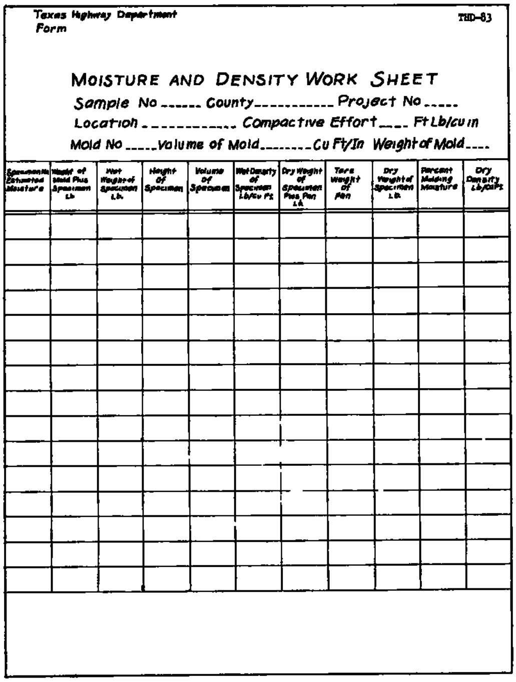
Prepare the material according to the procedure given in THD-53, Part II.

Compactive Effort

Compactive effort is defined as the total energy, expressed in ft.-lb., delivered by the compaction ram to each cubic inch of specimen. As proposed in this procedure, the modified hammer (weight 10-lb. fall 1.5 ft.) is used to compact layers 2 inches thick and 6 inches in diameter; 25 blows per layer will represent a compactive effort of 6.63 ft. lb. per cu. in. (25 x 10 lb. x 1.5 ft.) divided by (3 in. x 3 in. x 3.1416 x 2 in.).

In selecting a compactive effort for running triaxial tests, THD-80, it is important to choose the one that will produce

Figure C.



Texas Highway Department Form THD-83

MOISTURE AND DENSITY WORK SHEET

Sample No ------ County ----------- Project No -----

Location ------------ Compactive Effort ---- Ft Lb/cu in

Mold No ----- Volume of Mold -------- Cu Ft/In Weight of Mold ----

Specimen No. Estimated Moisture	Weight of Mold Plus Specimen Lb	Wet Weight of Specimen Lb.	Height of Specimen	Volume of Specimen	Wet Density of Specimen Lb/cu Ft	Dry Weight of Specimen Plus Pan Lb.	Tare Weight of Pan	Dry Weight of Specimen Lb.	Percent Molding Moisture	Dry Density Lb/cu Ft

Figure D.

the degree of compaction required by the conditions of the proposed use. The usual compactive efforts shall be as follows:

1. For flexible base materials and select soils with little or no tendency toward shrinkage or swell, use 50 hammer blows per layer (13.26 ft. lb. per cu. in.) There are special cases where this degree of laboratory compaction will not produce densities and strengths which are as high as those produced with rollers. In cases where this is strongly evident and the strength of the material when tested at 50 blows per layer compaction method is lower than appears reasonable, it is suggested that 75 blows per layer be used. In such cases field density should be controlled by testing so that it will be as great or greater than obtained by use of 75 blows per layer.

2. For moderately active soils use 25 blows per layer (6.63 ft. lb. per cu. in.). Most moderately active soils will have a PI between 20 and 35 if practically all particle sizes pass the No. 40 sieve. Most granular, select materials with PI's between 20 and 35 should be molded in accordance with step 1.

3. For very plastic clays, exhibiting high volume change, use 15 to 20 hammer blows per layer (4 or 5 ft. lb. per cu. in.).

4. Clean, cohesionless sands are exceptions and require special treatment. Specimens representing field conditions can be compacted by use of the proposed equipment if the hammer is converted to have a twin striking face and weighing 10 lb. It is also necessary to use a live rubber pad ½-inch thick to convert some of the impact into vibrations. The only other modification in technique is that the use of approximately 100 blows per 1-inch layer to produce the desired densities is indicated.

5. For selection of compactive efforts for soils stabilized with cement, asphalt and lime, see THD No. 96, 98 and 82.

6. For control of compaction by rollers, see THD-110.

Test Procedure

I. Calculation of Mix for Individual Specimens.

1. Estimate the weight of air-dry material that will, when wetted and compacted, almost fill the 6" x 8" mold.

2. Using the weight estimated in (1), compute the accumulative weights of the various sizes of the material based on the percentages obtained in the preparation of the large sample.

3. Weigh up the sample as calculated in (2), keeping the minus No. 10 or No. 20 mesh material separate.

II. Mixing of Specimen:

1. Place the aggregate portion (plus No. 10 mesh material), weighed out in

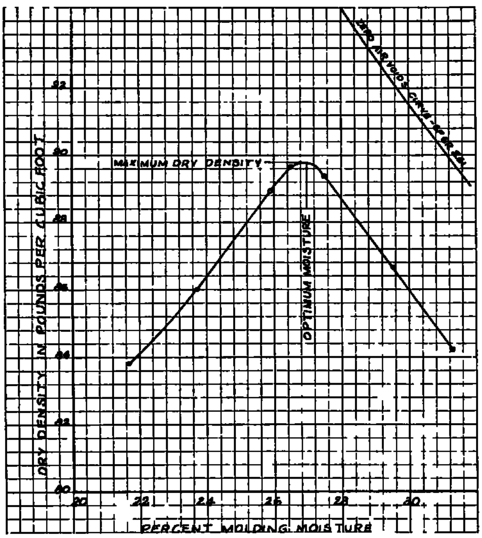


Figure E. Moisture-density curve.

step 1, in the mixing pan and add a known amount of water (a percentage based on the dry weight of the total material) pounds of water to be added equals pounds of water desired minus hygroscopic. Mix thoroughly with a trowel, breaking up the soft slakeable lumps of material. Do not crush any material that would not ordinarily slake.

2. Add the minus No. 10 mesh material to the wet aggregate and mix until uniform. Try not to lose any of the material. Cover mixture to prevent loss of moisture.

III. Molding of Specimens

1. The ten pound ram of the compactive machine should be adjusted to drop 18 inches.

2. The material is now molded in four equally weighed layers at the number of blows per layer as specified under "Compactive Effort." In case of harsh aggregate or material with low amount of fines, some of the fines out of the first layer should be placed on the bottom of the mold to insure a smooth bottom on the specimen.

3. After the last layer is compacted, the specimen is placed on the extra base plate and finished with the various hand tools, such as putty knife, plastic mallet, heavy lead hammer and circular plate with a plane surface. The surface of the specimen is checked by means of the small carpenter level to see if it is plane and level with respect to the top of the mold. Do not trim the top of the specimen.

4. Remove the mold from the small base plate, weigh and measure height of specimen. Record test data on form shown in Figure D. If the specimen is not of the right height, it can be easily corrected by weighing out air-dry material =

$$\frac{\text{amount used}}{\text{height obtained}} \times \text{height desired.}$$

5. Place a porous stone on bottom of mold and extrude specimen from mold, taking care not to lose any of the material.

6. Extrude specimen and if triaxial tests are not being run, break up material by hand and dry at 230 F.

7. Vary the amount of molding water, and repeat above operations in order to obtain several points for the moisture-density curve.

IV. Moisture Density Curve

The dry density of each specimen is calculated as follows:

1. Volume of specimen = height of specimen x vol. per linear inch (see "Calibration of Mold").

2. Wet weight of specimen = wet wt. of spec. and mold minus weight of mold.

3. $$\text{Wet density} = \frac{\text{wet wt. of specimen}}{\text{vol. of specimen}}$$

4. $$\text{Dry density} = \frac{\text{wet density of specimen}}{1 + \frac{\text{\% molding moisture}}{100}}$$

5. Zero air voids density =

$$\frac{\text{specific gravity} \times 62.5}{1 + \frac{\text{sp. gr.} \times \text{\% moisture}}{100}}$$

6. Plot dry density and zero air voids density against molding moisture (see Figure E). Where the specific gravity is not known, the value of 2.65 may be used as an average specific gravity for the majority of materials.

USE OF COMPACTION RATIO FOR SELECTION OF DENSITIES FOR SUBGRADES AND FLEXIBLE BASES

TEXAS HIGHWAY DEPARTMENT

THD-110

Foreword

Throughout this discussion the term "density" or "dry density" of any given soil mass, regardless of its moisture content, will refer to the calculated weight of dry soil per cubic foot. The "wet density" will include the weight of soil plus water in pounds per cubic foot. All moisture contents are calculated on the basis of oven dry weight of soil (dried to constant weight at 105 C.).

The fact that the same compactive effort produces different densities for the same soil when compacted at various moisture contents has been discovered independently by several sources, but Mr. R. R. Proctor worked out the first widely

TABLE A

Lab. No.	Material	Location	D_L	D_D	D_A in Lab Tests	Compaction Ratio (CR) Calculated**	Compaction Ratio (CR) From Fig. 3 $58 + \frac{D_L}{3.9}$	LL	PI	SL	SR	Soil Binder %	Triaxial Strength Class
49-1-R	Crushed stone	Servtex New Braunfels, Texas	126	141	139.5*	89.9	90.2	21	7	13	2.0	32	1
50-69-R	Gravel	Hybla Valley, Va.	122	138	136.5*	90.4	89.5	22	7	15	1.8	21	3.3
53-1-R	Light Caliche	Lubbock, Texas	85	101.5	98*	78.8	79.8	63	22	45	1.2	13	3.2
39-7-MR	Sand Clay	Austin Co. Texas	107+	127	124*	85.0	85 4	18	3	15	1.8	97	3.5
51-51-R	Clean Fine Sand	Canadian River Sand Roberts Co. Texas	103	112	110.5	83.4	84.3	21	0	21	1.65	100	-
51-112-R	Silt	Idaho WASHO Test Section	77.5+	100	95*	79.5	77.9	39	13	23	1.56	99	3.5
39-11-MR	Black Gumbo Clay	Manor, Texas	56+	106	92	72.0	72.4	74	45	10	2 0	100	5.0

* Materials compacted to densities equal to those obtained from normal amounts of rolling by employing a laboratory compactive effort of 13.26 ft. lb. per cu. in. See THD-83. The Canadian River sand was compacted by the use of a special technique to obtain vibration from drop hammers as outlined in THD-83. The black clay 39-11-MR was found to have best strength when compacted to this density by use of 6.62 ft. lb. /cu. in. and subjected to capillary wetting. This density was also found to exist underneath old pavements on this type of soil (see Table B)

** $CR = \left(\frac{D_A - D_L}{D_D - D_L}\right) 100$

+ D_L calculated from the formula $D_L = \frac{SR \times 62.5}{1 + \frac{LL - SL}{100}(SR)}$ All other D_L values were obtained from rodded unit weight tests

recognized test for determining the optimum moisture content of the soil and checking it in the field. His tests were made in connection with the building of earth dams, and the compaction obtained in the Proctor test mold was presumably equivalent to that obtained by the sheep's foot rollers used on these first jobs.

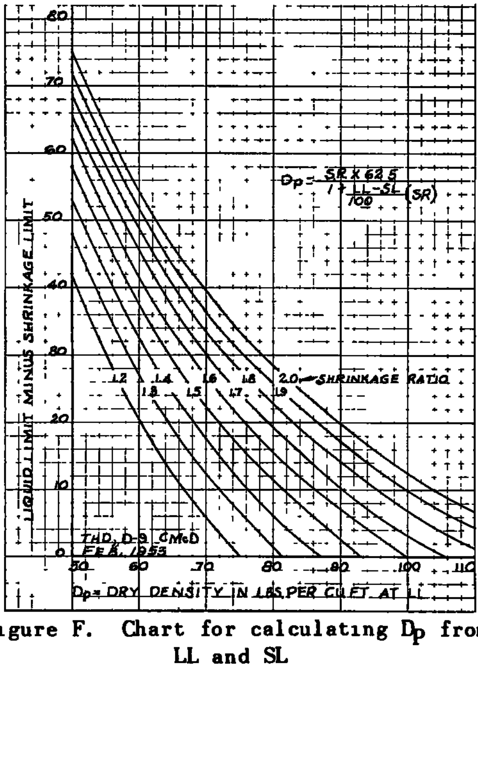


Figure F. Chart for calculating D_P from LL and SL

These tests are based on the fact that for each soil there is but one moisture content, termed the optimum moisture content, at which the maximum density is produced by a specific degree of compaction. Our triaxial test studies relative to thickness and quality of flexible bases have thoroughly convinced us of the need for a method of density selection which can be applied successfully to widely varying types of soils. In order to do this it is recommended that the method described herein which is based upon "Compaction Ratio" be used. In its simplest terms compaction ratio is defined as the ratio of the difference between actual and loose densities to the difference between the dense and loose densities and is expressed in percentage as follows:

$$CR = \left(\frac{D_A - D_L}{D_D - D_L}\right) 100$$

Where:

CR = Compaction ratio in percent

D_L = Loose Density (Dry Unit Weight in Lbs. per Cu. Ft.)

D_A = Actual Density to be obtained (Dry Unit Weight in Lbs. per Cu. Ft.)

D_D = Densest condition the material can be compacted to by using 30 ft. lbs./cu. in. compactive effort in the laboratory. See THD-Procedure 83.

The terms "Compaction Ratio" and "Relative Density" should not be confused because CR must be multiplied by the ratio of D_D/D_A to obtain "Percent Relative Density". The term "Relative Density" is based upon a concept of filling voids with solids and is used frequently in soil mechanics and its use for control of compaction of subgrades and base materials was proposed in an interesting article by Professor Donald M. Burmister and published in Volume 48 of the Proceedings of the ASTM.

"Compaction Ratio" expresses the degree to which it is desirable to compact soil materials on the basis of the compactibility of the material; namely, the difference between the dense and loose densities (D_D - D_L). This method is preferred in lieu of the usual method of specifying an arbitrary percentage of some maximum density. The method of specifying percent density is not sound because it is a percentage of a variable density which has an important influence upon strength. As an example, assume that a base material can be compacted in the laboratory to a maximum density D_D and 90% density is required. The density so specified can usually be obtained by lightly rodding the dry material into a one-half cu. ft. bucket. It is obvious that when such a material is at such a low density that its strength is only a fraction of what it should be. If the same minimum density of 90% of D_D for some high volume change soils is used, a condition of overcompaction could be attained. Therefore, various soil materials should be compacted to different percentages of their maximum densities. The desirable percent density to use can be predetermined from strength and volume change tests but they are too time consuming to be used during construction. By use of the proposed method the compaction

TABLE B

Lab No	County	Highway Number	Description of Material	Density lb./cu. ft. Dry: Field	Field: No. Tests	Lab. Compactive Effort Ft. lb./cu. in: 6.63	13.26	19.89	Before Rolling: PI	Before Rolling: % S.B.
49-4-R	Lampasas	190	Caliche and Limestone with Soft Sandstone	126.8	1		127.0+		7	41*
R-1 thru R-10	Williamson	79	Clay-Gravel Plus Waste Lime	128.9	5		128.2+		8	28
L-1 thru L-5	Llano	29	Granite Gravel Plus Waste Lime	128.0	10		127.6+		11	25
48-29-R	Bexar (San Antionio Expressways) UGI 1083 (5)		Crushed Rock Subbase from Rabke Pit	129.7	48		128.0+	128.9	12	27
48-40-R	"		Crushed Rock Base from McDonough Pit	137.1	34		133.5+	134.5	8	29
48-21-R	"		Leon Creek Gravel-Sand Admix Subbase	142.4	32		141.9+	142.7	13	31
39-11-MR	Travis	20	Black Clay (Gumbo)	92.0**	3	92.0***+	98.5	102.2	45	98
47-471E	Williamson	Old U.S. 81	Black Clay	85.0**	3	85.0+			37	97
47-136-E	Bastrop	St. Hwy. 20	Sand Clay	115.8	15	111.5	114.9+		6	98
47-143-E	"	"	" "	118.9	14	117.7	121.4+		14	95
47-158-E	"	"	" "	117.1	3	112.7	116.5+		14	96
49-14-R	"	"	Crushed Stone	126.0	4		125.0+		6	28

* Lab. specimens and road samples contained from 48 to 50 percent soil binder after compaction, showing a similar breakdown of particle sizes for field and laboratory operations

** Samples from old road subgrade after clay had consolidated or swelled so as to come to its final density.

*** Standard AASHO density = 85 lb./cu. ft.

+ Values obtained in same manner D_A values were obtained in Table A.

ratio and the minimum desirable density can be predetermined from simple unit weight and compaction tests which will be explained under procedure. The procedure for laboratory compaction (THD-83) used in determining D_D provides for compacting coarsely graded soils and base materials without having to split the sample over such screen sizes as $\frac{1}{4}$ in. or No. 4, etc. There will be instances where it will not be practical to use the proposed method. Some of these are as follows:

1. Where soils are considerably wetter than optimum and it is not practical to aerate them.

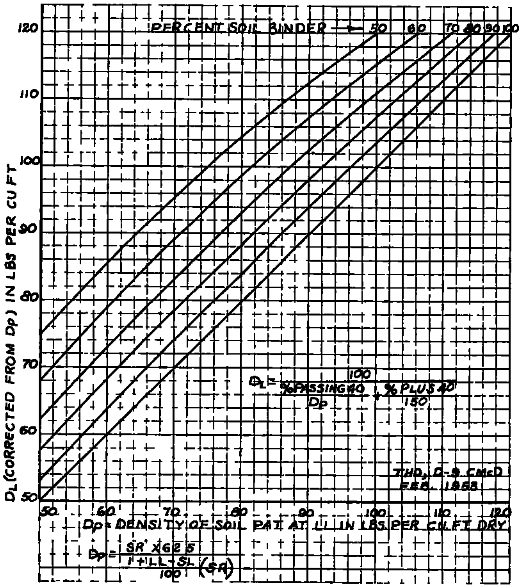


Figure G. Chart for calculating D_L from D_P.

2. Where high fills (above 15 ft.) are being constructed out of high volume change soils. In this case higher densities and less moisture than proposed should be used because the surcharge weight of the fill restricts excessive swellage and less settlement will be obtained.

Procedure

1. Secure and prepare samples in accordance with THD-53, Part II.

2. Determine loose density D_L by one of the following methods:

A. For all soil materials containing small amounts of plus No. 40 mesh material except clean sands.

(a) Determine the dry unit weight Dp of the soil at the LL condition from the formula

$$Dp = \frac{SR \times 62.5}{1 + \frac{LL - SL}{100}(SR)}$$

Where:

SR = Shrinkage Ratio
SL = Shrinkage Limit
LL = Liquid Limit

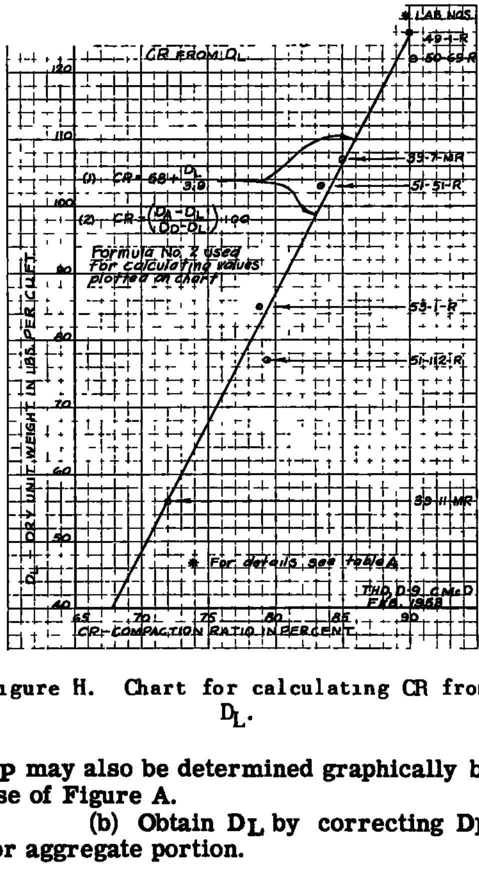


Figure H. Chart for calculating CR from D_L.

Dp may also be determined graphically by use of Figure A.

(b) Obtain D_L by correcting Dp for aggregate portion.

$$D_L = \frac{100}{\frac{\%\ \text{pass}\ \#40}{Dp} + \frac{\%\ \text{Ret. on}\ \#40}{62.5\ (\text{bulk gravity of portion ret. on}\ \#40)}}$$

In cases where bulk gravities of plus #40 portion varies between 2.3 and 2.5, Figure B may be used. Bulk specific gravity is determined by THD-74.

B. For materials containing predominant amounts of plus #40 material and clean fine sands passing the #40 mesh sieve.

(a) Run rodded unit weight on total air-dry material in accordance with ASTM

C 29-42. Briefly, this procedure is as follows: Rod material in three layers at 25 blows per layer, in a standard one-half cu. ft. unit weight bucket with the blunt end of a 5/8 in. diameter steel rod 24

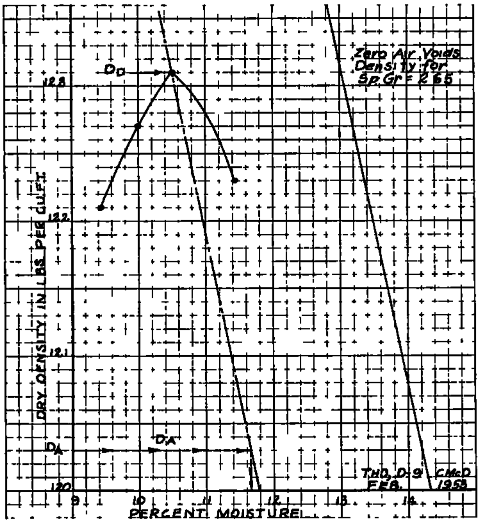


Figure I. Chart for estimating compaction moisture content.

in. long. The material is struck off with the rod and its unit weight in lbs. per cu. ft. calculated. This is D_L.

3. Enter D_L on Figure C to determine CR or calculate by the formula

$$CR = 58 + \frac{D_L}{3.9}$$

A comparison of CR values obtained by this method to those obtained from tests are shown in Table A.

4. Run compaction test for moisture-density relations in accordance with THD-83 using 112 blows per layer. Maximum density at optimum moisture is D_D.

5. Calculate D_A, the actual density desired, by the following formula:

$$DA = \frac{CR}{100}(D_D - D_L) + D_L$$

The usefulness of the above formula was checked by: (1) selecting a laboratory compactive effort which produces densities comparable to normal amounts of rolling in the field as shown in Table B; (2) using this laboratory compaction procedure to determine D_A values shown in Table B. These data indicate that all soils materials except swelling clays could and should be compacted to a minimum density equal to or greater than D_A. High volume change clay soils (PI above 35 and containing little or no plus #40) should not be compacted to a density in excess of D_A and can be satisfactory when compacted to a density of as low as seven pounds/cu. ft. below D_A.

6. Estimate optimum moisture content for rolling as follows:

(a) Plot the optimum moisture vs. maximum density obtained in step 4 of each of these soils on a sheet similar to Figure D. The "absolute specific gravity" of each soil can be determined by test (THD-73) and the correct zero air voids density curve drawn or an assumed Sp. Gr. of 2.65 can be used with satisfactory accuracy.

The zero air voids density curve can be plotted from the following equation:

$$\text{Zero Air Voids Density} = \frac{\text{Sp. Gr.} \times 62.5}{1 + \left(\frac{\%\ \text{moisture}}{100} \times \text{Sp. Gr.}\right)}$$

(b) Draw a line of equal air voids passing through D_D and parallel to the zero air voids line. See dashed line in Figure D.

(c) This line intersects D_A at a desirable moisture content for compaction. This moisture content may need to be varied for various rollers and the number of passes they make and thicknesses of lifts being rolled. High volume change clay soils should be compacted when containing at least as much moisture as determined in this step but should not contain more than 5 percent above this minimum. In order to materially reduce heaving and dry weather cracking, the compaction moisture for such soils should always be permanently preserved by whatever means is economically feasible. This can usually be accomplished by placing bituminous membranes or granular materials on top of the final layer of any series of layers placed during one fairly continuous series of operations.

Concluding Remarks

It probably will expedite matters to select a few typical samples representative of each of the large deposits or general types of materials to be encountered, and run the necessary tests to determine D_D and D_L for each material prior to the start of construction. Additional tests for determination of D_D and D_L can and should be made during construction especially when materials are encountered which have not been tested. The inspector who will control this part of the project should familiarize himself with the "looks" and "feel" of each of these soils at their optimum moisture content in order that he can do part of the inspection for moisture control in the field by observation. So long as the specific gravity and soil constants for a given material do not vary appreciably, D_D and D_L will remain fairly constant for wide variations in gradation. At the start of the project the effectiveness of the number of passes of a given roller used on a given depth of layer containing optimum moisture content (step 6) should be checked by making in place density tests (see THD-85). If these densities meet the requirements for D_A, discussed in step 5, this rolling procedure can be used for compacting the remainder of this material. If this rolling procedure produces densities lower than D_A, any one of a number of changes will increase the densities from rolling. Some of these are as follows: (1) increase the number of passes; (2) increase inflation pressures and possibly the weight of the roller; (3) vary the moisture content; (4) reduce the thickness of lift. Throughout the progress of the job, in place density tests should be made immediately after rolling and the moisture content and density compared to those required.

This procedure calls for an amount of rolling which will compact a material to a desirable state for subgrade and base construction. For purposes of densification control, the condition of the soil materials as proposed herein should be specified in lieu of trying to accomplish the same end by paying for hours of rolling. This is because better control will be obtained through testing and once contractors are told what moisture-density conditions are desired, they will find the best and most economical ways of obtaining them. The amount of rolling required to obtain minimum density requirements may not be adequate for finishing purposes, therefore, the amounts of rolling hours for finishing probably should be handled as a separate item.

Discussion

W. H. CAMPEN, Manager, Omaha Testing Laboratories — McDowell's paper involves a consideration of the principles pertaining to the densification of soils. He claims that the general practice of specifying percentage of maximum laboratory density in the compaction of soils is not a satisfactory criterion for two principle reasons: (1) because inadequate strength is developed in cohesionless mixtures and (2) because unstable conditions are developed in highly plastic mixtures. Due to these objectionable features, he has worked out a method whereby desirable densities may be determined and specified for various soils. These densities are not directly related to maximum laboratory values and are so selected as to produce maximum strength consistent with durability.

I certainly agree that the same degree of compaction does not give desirable results with all soils. Furthermore, no one can criticize his method of obtaining desired results. However, the same results may be obtained by use of the standard methods. For instance, the percentage of relative density may be varied to suit different soils. Also the compactive effort of the standard methods can be varied as has been done by the U. S. Engineers.

In a general way McDowell indicates that certain densities are required, in the field, for different soils in order to obtain satisfactory performance. Why not then determine these densities and use them as a basis of preparing specifications.

CHESTER McDOWELL, Closure — In order to carry out Campen's suggestion of using densities as a basis of preparing specifications, it would be necessary to develop a tabulation showing the densities required for all variations of subgrade soil and subbase and base materials to be encountered. Such an approach becomes impractical when various formations are encountered, especially when a contractor may use from any of a number of possible sources of flexible base materials existing within a 50-mile radius of a given project. Campen may believe that experience with various compaction equipment has established the values of densities necessary for good construction. However, the author feels that it is doubtful if we know enough about compaction to be able to specify densities in pounds per cubic foot necessary for all soil materials encountered on any one given project. It would seem desirable to be able to have a method whereby the intent and purpose of the specifications can be checked on the job every time necessary and especially when the nature of the material being compacted changes unexpectedly. The use of compaction ratio equal to 58 plus $D_L/3.9$, in which D_D and D_L are determined by tests in the laboratory and D_A is obtained from in-place density tests, as a specification appears to offer the most ideal method of securing good compaction.

In the case of clay soils it is desirable to consider $58 + D_L/3.9$ as a maximum and values of 5 to 10 lower should be acceptable. In the case of all other soils, especially granular base materials, $58 + D_L/3.9$ should be considered a minimum requirement for good compaction.

Relationship Between Density and Stability of Subgrade Soils

H. B. SEED, Assistant Professor of Civil Engineering, and
CARL L. MONISMITH, Instructor in Civil Engineering,
University of California

This paper discusses some of the limitations of the concepts that the stability of a compacted soil increases with an increase in density and that every effort should be made to attain the highest practical density in field compaction. Comprehensive test data on the relationship between density, stability, water content, and degree of saturation for two soils compacted by kneading action are presented, together with a description of the Triaxial Institute kneading compactor. In the tests, stability is measured by triaxial-compression tests and by the Hveem Stabilometer. The significance of the criterion of stability adopted with respect to the relationship between density and stability is discussed. The relationship between the attainable stabilities and those determined by specifications based on the standard Proctor and modified AASHO compaction tests is demonstrated.

Data are also presented on the density-versus-stability relationships of soils compacted by impact and static procedures, and the effect of compaction method on the density-versus-stability relationship is shown. The conflicting conclusions which may be reached on the basis of tests on samples prepared by static and impact methods or by static and kneading methods are indicated.

SINCE the principles of soil compaction were first described by R. R. Proctor (1) some 20 years ago, the use and interpretation of soil-compaction tests have changed relatively little. Though new compaction procedures have been developed, tests are still conducted to determine the optimum water content at which a given compactive effort will produce the maximum density of the soil. This water content and density are then used as criteria for field compaction of earth fills and pavements.

In adopting these criteria, it is implicitly assumed that density is a measure of the desirable characteristics of the compacted soil, such as strength or stability, compressibility, and sometimes permeability. A recent article (2) states: "This optimum condition, producing a maximum density for the given compaction method, is generally the strongest and most permanently stable condition for the soil resulting from the particular compaction procedure." Properly interpreted, this statement is no doubt true, but its validity will depend on the conditions to which the compacted soil is exposed. There is considerable evidence (3, 4, 5, 6, 7) that maximum strength and maximum density of a soil, even for the same compaction method, are not necessarily attained at the same time; this evidence has been obtained in both laboratory and field tests. Nevertheless, many engineers seem to believe that the higher the density to which a soil is compacted, the greater will be its strength and stability and that every effort should always be made to obtain the highest possible density. It would seem pertinent, therefore, to discuss some of the limitations of this concept as indicated by the results of recent laboratory tests. Such a discussion is particularly desirable now that pavements are being designed for conditions other than complete saturation (8); an understanding of the relationship between density and stability would seem to be essential for the intelligent design of such pavements.

LABORATORY COMPACTION TESTS

The object of any laboratory compaction test is to reproduce in the laboratory the compaction effects produced by equipment in the field. A variety of procedures are in use at the present time. In most tests, the soil is compacted by dropping a falling weight on to the surface of the soil, a process referred to as "impact compaction." In some cases, soil is compacted by subjecting it to a static load, which is built up slowly to some predetermined value and then released, a process referred to as "static compaction." The fact that these two methods of compaction result in density-water-content curves having different forms and that they produce soil specimens having quite different stress-strain characteristics has long been recognized (3).

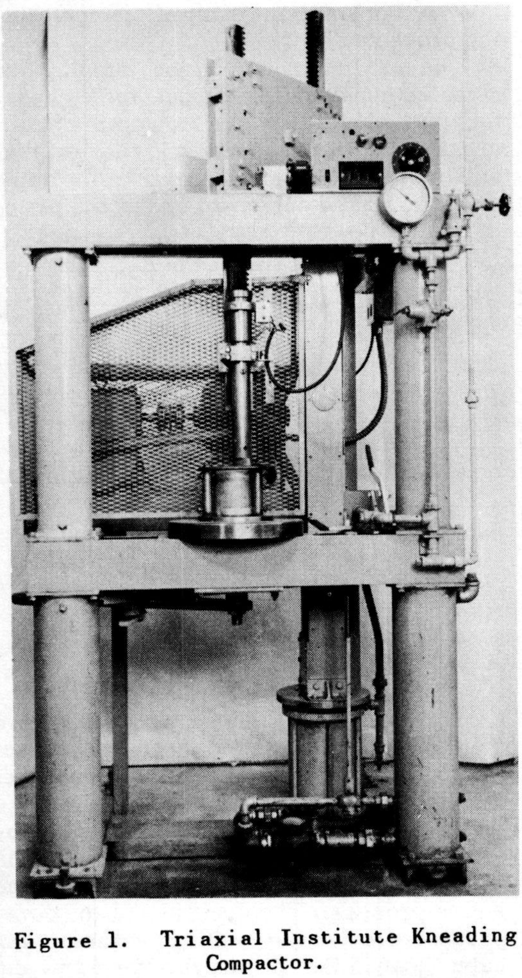


Figure 1. Triaxial Institute Kneading Compactor.

Extensive studies of soils compacted in the field by sheepsfoot and rubber-tired rollers have also shown that the stress-strain characteristics of those soils are different from those prepared in the laboratory by either static or impact compaction. Since laboratory-prepared

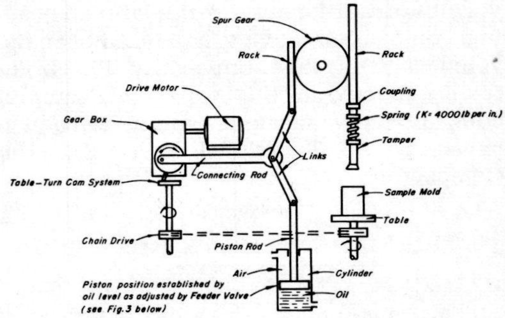


Figure 2. Compactor mechanical tamping system.

specimens are used for design purposes, in recent years considerable effort has been devoted to the development of laboratory procedures which will satisfactorily duplicate the effects of field compaction.

These efforts have resulted in a compaction method which more closely simulates the effect of sheepsfoot or rubber-tired rollers. The action of this equipment is to build up the pressure on a small area of soil to a definite value, maintain this pressure for a small element of time, and then gradually reduce the pressure; this method of load application has been termed a "kneading action." It has been shown that the compaction of soil specimens by this method offers good possibilities for the satisfactory preparation of laboratory samples having properties sufficiently close to those of the soil compacted in the field.

A laboratory kneading compactor has been developed at Northwestern University (2) and, on behalf of the Triaxial Institute, a model originally designed and used by the California Division of Highways has been modified and developed at the Uni-

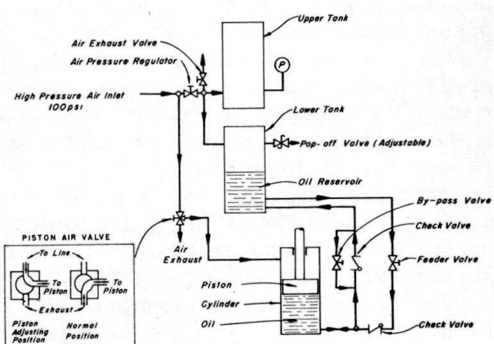


Figure 3. Compactor hydropneumatic control system.

versity of California. A miniature kneading compaction device has also been developed at Harvard University (9). In the tests reported in this paper, all samples compacted by kneading action were prepared by the Triaxial Institute Kneading Compactor.

TRIAXIAL INSTITUTE KNEADING COMPACTOR

This equipment, a view of which is shown in Figure 1, may be used for compacting soil or asphalt samples. It is a mechanical device for applying a series of tamps of fleeting pressures by means of a tamping foot to a sample contained in a cylindrical mold. The shape of the tamping foot is approximately that of a segment of a circle having the same diameter as the forming mold; its area is approximately a fourth that of the cross-sectional area of the mold.

A schematic drawing of the mechanical tamping system, which employs a toggle-press principle, is shown in Figure 2. Power for the operation of the toggle-press mechanism is provided by an electric motor through a speed reduction gear, flywheel, and connecting rod. The action is such that in any one tamp the pressure is gradually built up and then allowed to dwell on the sample for a fraction of a second before being released. The compaction rate is 30 tamps per minute.

The overall dimensions were selected to provide ample space for compacting specimens up to 6 inches in diameter and 12 inches in height.

In order to control the pressure exerted on the test specimen by the tamper, a combination hydraulic-pneumatic control system is used. A schematic drawing of this system is shown in Figure 3. Air from a high-pressure line passes through a pressure regulator, which can be set at any predetermined value, into the upper portion of the oil reservoir. This reservoir is situated in the pipe column which also serves as a member of the compactor frame. A feeder valve controls the flow of oil into the cylinder containing the piston, which is attached to the lower link of the press. This feeder valve is used to adjust the height of the tamper in the mold prior to the start of the compacting procedure.

The maximum load on the tamping foot remains constant throughout the compaction procedure since, as soon as the piston exerts more pressure on the oil than exists in the compressed air in the tank, oil is squeezed out from under the piston through a one-way check valve back into the oil reservoir and a pop-off valve, which is set at the predetermined pressure, allows excess air to escape. To ensure that the full pressure will always be applied to the sample, a bypass valve is kept open a certain amount during the entire process. The pressure of the compressed air and the setting of the bypass valve govern the pressure exerted by the tamper on the sample.

Before the compactor is used, it must first be calibrated by measuring the load exerted by the tamper foot for different air pressures and settings of the bypass valve. This is done by inserting a dynamometer above the tamper foot and obtain-

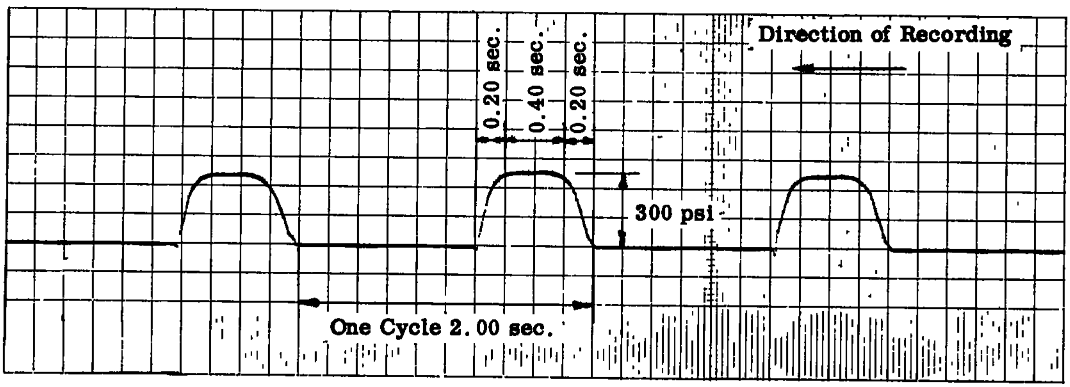
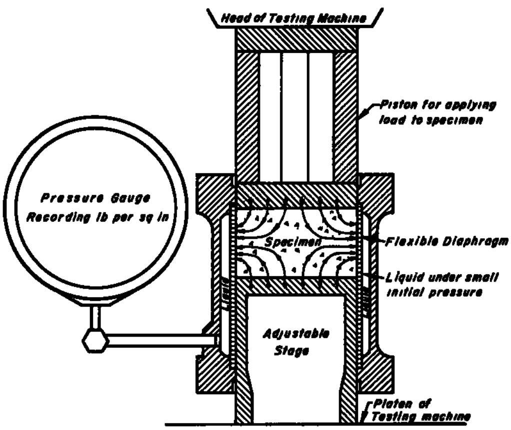


Figure 4. Typical oscillogram showing load vs. time relationship for the Triaxial Institute Kneading Compactor.

ing oscillograms of the relationships of load versus time. A typical oscillogram obtained in this way during compaction of a silty clay is shown in Figure 4. It will be seen that the pressure-versus-time curves consist of three distinct parts: first, the application, second the dwell, and third, the release.



Note:
Specimen given lateral support by flexible side wall which transmits horizontal pressure to liquid Magnitude of pressure may be read on gauge

Figure 5. Hveem Stabilometer.

STABILITY

The stability of a soil may be broadly defined as the load or pressure which it can support without excessive deformation. Exactly what constitutes excessive deformation is a matter of opinion. Because of the difficulty of measuring stability in the field a variety of laboratory methods for measuring stability have been developed and correlated with the performance of soils underlying pavements. Thus these test methods provide fairly reliable means for measuring the relative stabilities of soils.

Probably the most widely used index of the stability of a subgrade soil is the California Bearing Ratio which is based on the resistance to penetration of the soil by a plunger having a base area of 2 sq. in.

In California, the index of stability used for design is the resistance value as measured by a Hveem Stabilometer test (10). This is a closed-system, triaxial-compression test, as shown in Figure 5, using a specimen 4 inches in diameter and about 2½ inches high. The vertical load on the specimen is applied at a constant rate of strain (0.05 inches per minute) while the pressure is allowed to build up in the liquid cell which encircles and confines the specimen laterally. The lateral pressure, P_H, transmitted through the specimen when the vertical pressure, P_V, is 160 psi. is recorded. An indication of the surface roughness of the specimen is then obtained by determining the displacement, D, of the specimen, and the resistance value or stabilometer R value is computed from the formula

$$R = 100 - \frac{100}{\frac{2.5}{D}\left(\frac{P_V}{P_H} - 1\right) + 1}$$

The R values determined in this way have been correlated with the service behavior of soils under pavements and have been found to be a satisfactory measure of relative stabilities.

A third test which may be used to measure the relative stability of a soil is the triaxial compression test, performed under constant lateral pressure. Either the ultimate strength of a test specimen or the modulus of deformation at a particular strain may be used as an index of stability.

In the tests reported in this paper, all three of the above methods were used. The methods are used, however, only to compare relative stabilities of samples, and no attempt is made to compare the values obtained by the various methods.

RELATIONSHIP BETWEEN DENSITY AND STABILITY OF SOILS COMPACTED BY KNEADING ACTION

Since kneading compaction best simulates the effects of compaction equipment, it was considered desirable to determine the relationship between density and stability for samples compacted by this method. A comprehensive investigation was made using a silty-clay soil (liquid limit = 37, plastic limit = 23) from Vicksburg, Mississippi. A series of tests were made to establish the relationships between density and water content and between strength or modulus of deformation and water content for the soil compacted at each of six different compactive efforts.

For tests at any one compactive effort,

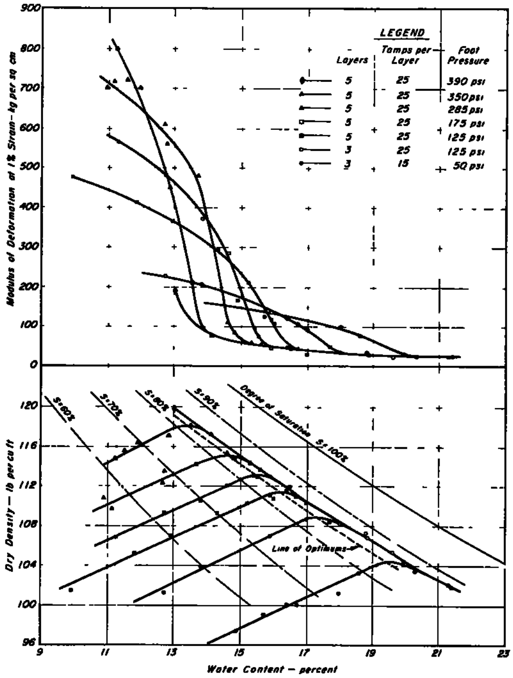


Figure 6. Relationship between water content, dry density, and modulus of deformation at 1% strain.

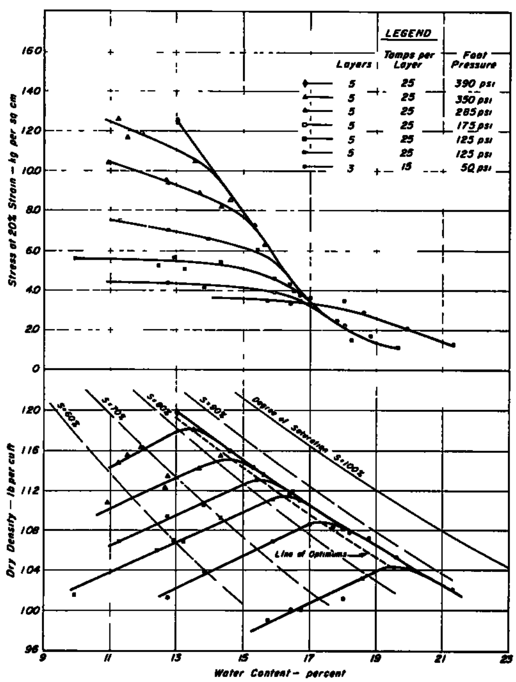


Figure 7. Relationship between water content, dry density, and stress at 20 percent strain.

the soil was first oven dried and samples were then mixed at about six different water contents. Each sample was placed in a sealed container and allowed to condition for one day prior to compaction. The samples were then compacted in 6-inch-diameter molds to form specimens $4\frac{1}{2}$ inches high. The compacted samples were used to determine the relationship of dry density versus water content in the usual manner. After the weight and vol-

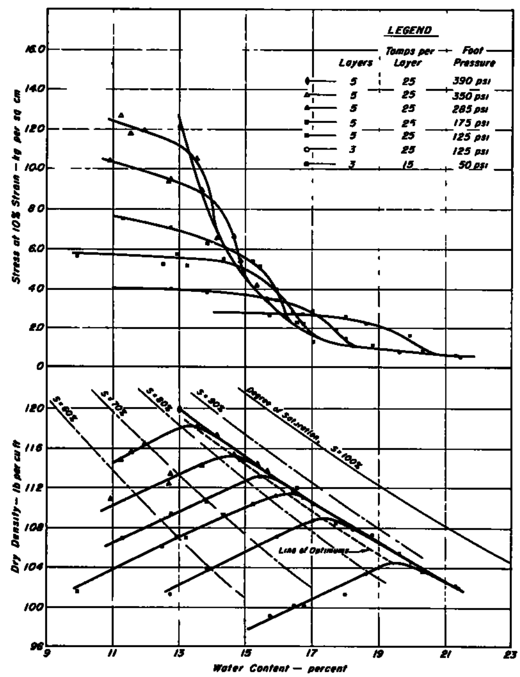


Figure 8. Relationship between water content, dry density, and stress at 10 percent strain.

ume of a compacted sample had been measured and a sample had been taken for water-content determination, a specimen was cut from the sample having a diameter of 1.4 inches and a height of about 4 inches.

This specimen was subjected to a triaxial-compression test of the unconsolidated, undrained type, using a confining pressure of 1 kg. per sq. cm. Load was applied at the rate of 8 kg. per min. until failure, and the stress-versus-strain relationship for the specimen was determined. In this way the water content, density, strength, and deformation characteristics of each compacted sample were obtained.

The results of these tests are shown in Figures 6, 7, and 8. Figure 6 shows the density-versus-water-content curves and the modulus of deformation at 1 percent strain versus water content for each compactive effort. It will be seen that for each compactive effort the modulus of deformation is low on the wet side of optimum but that it begins to increase as the optimum water content is approached and, for the range of water contents used in these tests, continues to increase with decreasing water content on the dry side of optimum even though the density of the samples is decreasing.

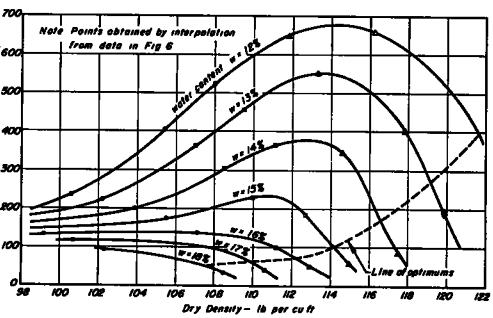


Figure 9. Relationship between dry density and modulus of deformation at 1% strain for constant water contents.

The significance of these results is best seen from the relationship of modulus of deformation, density and water content as shown in Figure 9. These curves, showing modulus of deformation versus dry density at a series of constant water contents, were interpolated from the results in Figure 6. It will be seen that the effect of increased density on the modulus of deformation of the soil depends on both the water content and the range of densities considered. At a water content of 13 percent, an increase in dry density from 100 to 110 lb. per cu. ft. caused an increase in modulus of deformation from 190 to 470 kg. per sq. cm.; at a water content of 17 percent, however, the same increase in density caused a reduction in modulus of deformation from 120 to 50 kg. per sq. cm. Thus, a given increase in density may increase or decrease the modulus of deformation, depending on the water content of the soil.

Again, at a water content of 13 percent, an increase in dry density from 100 to 114 lb. per cu. ft. caused an increase in modulus of deformation from 190 to 560 kg. per sq. cm., but a further increase from 114 to 120 lb. per cu. ft. caused a reduction in modulus of deformation from 560 to 175 kg. per sq. cm. The effect of increased density may thus be to increase or reduce the modulus of deformation depending on the range of densities concerned.

Conditions under which an increase in density may cause a reduction in strength or deformation index are associated with the characteristic overlapping of the curves, in Figure 6, showing the relationship of this index to the water content

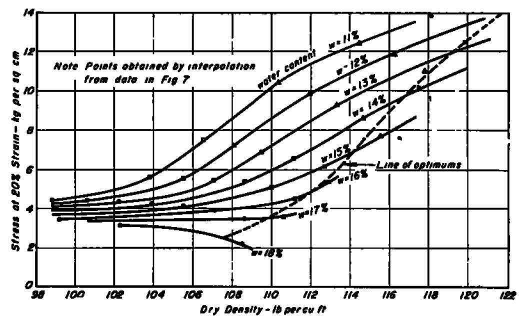


Figure 10. Relationship between dry density and stress at 20% strain for constant water contents.

of the soil.

The effect of dry density and water content on the strength of the compacted soil is shown in Figures 7 and 8. On the dry side of the optimum water content for any given compactive effort, specimens failed by shearing along a well-defined plane. At about the optimum water content and at higher water contents, specimens bulged considerably but continued to support increasing loads. For such specimens, failure is considered to occur when the deformation becomes excessive; it is often defined as the stress required to cause say 10 or 20 percent strain of the test specimen.

Figures 7 and 10 show the variation of strength with dry density and water content when failure is defined as the stress required to cause 20 percent strain. As for the modulus of deformation data, the curves in Figure 10 have been obtained by interpolation from the data in Figure 7. It is readily seen that within the range of densities investigated there is a marked difference between the effect of increasing

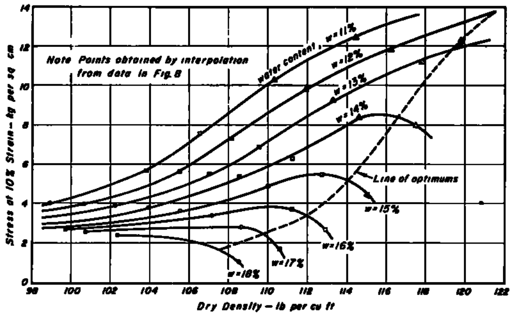


Figure 11. Relationship between dry density and stress at 10% strain for constant water contents.

density on strength and on modulus of deformation. Except for the high water content of 18 percent, an increase in dry density in all cases caused an increase in strength; this type of density-stability

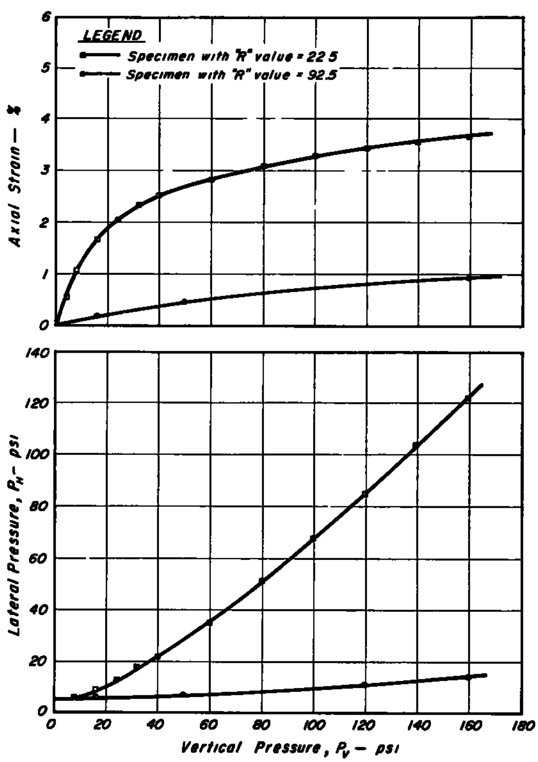


Figure 12. Typical results of stabilometer tests on specimens of silty clay with high and low stabilities.

relationship is associated with cases where there is no overlapping of the curves of stability versus water content for constant compactive efforts. The magnitude of the effect of increasing density, however, decreased with increasing water content. At a water content of 14 percent, an increase in density from 100 to 110 lb. per cu. ft. caused an increase in strength of about 50 percent, while at a water content of 17 percent, the same increase in density had practically no effect on the strength.

The test results when failure is defined as the stress required to cause 10 percent strain are shown in Figures 8 and 11. For these results, as for the modulus of deformation at 1 percent strain data, there is some overlapping of the curves of strength versus water content for constant compactive efforts, indicating that an increase in dry density may cause a reduction in strength. In this case there is more variation in form of the curves of strength versus density at constant water contents than for the curves in Figure 10. For the range of densities and water contents investigated, the strength increases consistently with density for water contents below 13 percent, but for water contents above 14 percent, the strength may increase or decrease with increasing density depending on the range of densities considered.

It will be seen from these results that the effect of density and water content on the stability of the soil depends on the criterion used to define stability. If the ultimate strength or the stress at 20 percent strain is used as a criterion of stability, then it may in general be true that stability increases with density. If the modulus of deformation at 1 percent strain or the strength as defined by the stress required to cause 10 percent strain is used as a criterion of stability, then for partially saturated soils, stability may

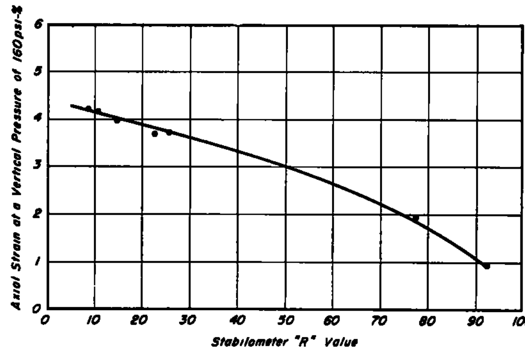


Figure 13. Relationship between stabilometer R value and axial strain at which R value was determined.

increase or decrease with increasing density. In tests which are customarily used as criteria of stability, the strain of the specimen at which the measurement is taken is not always clearly defined. For example, in the California Bearing Ratio test, the strain of the soil subjected to load at the time the measurement is made is probably of the order of 5 percent. In the Hveem Stabilometer test, the strain of the specimen at the time the stability is determined varies with the water content and stability of the specimen.

Typical results of stabilometer tests on specimens of the silty clay having high and low stabilities are shown in Figure 12. It will be seen that for the specimen having a resistance value of 92.5 the strain at which this value was determined was 0.9 percent, while for the specimen having a resistance value of 22.5 the strain was 3.7 percent. For the Vicksburg silty clay, the variation of the strain at which the resistance value was determined for specimens with resistance values ranging from 7 to 92.5 is shown in Figure 13; the strains range from 0.9 to 4.2 percent. Thus in both the CBR and the Hveem Stabilometer tests, the stability of the soil is determined at strains appreciably less than 10 percent; on the basis of the compression-test data, it would be expected that an increase in density would not necessarily lead to an increase in stability, therefore, for soils compacted by kneading action.

In order to obtain some idea of the condition to which this soil might be compacted in the field, standard Proctor and modified AASHO tests were conducted to determine the optimum water contents and maximum dry densities given by these compaction procedures. The tests gave the following results:

Compaction Test	Optimum Water Content	Maximum Dry Density
Standard Proctor	18 %	105 pcf.
Modified AASHO	14 %	117 pcf.

On the basis of the standard Proctor test, this soil might be compacted at a water content of 18 percent to a relative compaction of 100 percent, that is to a dry density of 105 lb. per cu. ft. At this water content, both the modulus of deformation at 1 percent strain and the strength of the compacted specimens, show a slight reduction with increase in density; at a relative compaction of 100 percent based on the standard Proctor test, the modulus of deformation and the strength are higher than can be attained at higher values of relative compaction.

If compaction is controlled by the modified AASHO test, the soil might be compacted at a water content of 14 percent to a relative compaction of 90, 95 or 100 percent depending on the specifications, that is, to a density between 105 and 117 per cu. ft. It will be seen from Figure 9 that at a water content of 14 percent the maximum value of the modulus of deformation is attained at a density of 113 lb. per cu. ft., that is, a relative compaction of 97 percent. However, a further increase in relative compaction to 100 percent causes the modulus of deformation to be reduced to only 33 percent of its maximum value. In the case of strength (for 10 percent strain), the strength increases with density up to a relative compaction of 99 percent with only a slight reduction occurring if the relative compaction is further increased to 100 percent.

These results illustrate the deleterious effects of overcompacting a soil as well as those of using too high a water content. At the optimum water content of the standard Proctor compaction test (18 percent), the maximum modulus of deformation at 1 percent strain was only 95 kg. per sq. cm., while at the optimum water content of the modified AASHO compaction tests (14 percent), the maximum modulus of deformation was 385 kg. per sq. cm. Again, at a water content 1 percent above the modified AASHO optimum, the maximum stability for a relative compaction of 95 percent was only about 65 percent of that attained at the same relative compaction at the optimum water content. The importance of careful water content control in obtaining the maximum stability of partially saturated soils cannot be overemphasized.

Examination of the data in Figures 6, 7, and 8 shows that for each compaction curve there is a marked change in stability at about the optimum water content. At water contents above the optimum the stability is low, while at water contents below the optimum the stability is relatively high. These results would seem to indicate that the lower values of stability are associated with higher degrees of saturation, rather

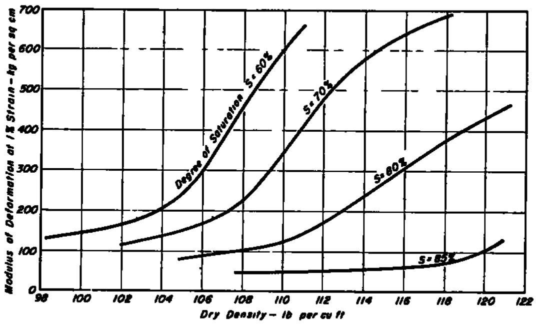


Figure 14a. Relationship between dry density and modulus of deformation at 1% strain for constant degrees of saturation.

than particular water contents, and that the degree of saturation may be a more-important factor in determining stability than the water content. Curves of modulus of deformation at 1 percent strain versus density for various constant values of the degree of saturation are shown in Figure 14a; these curves have been obtained by interpolation from the data in Figure 6. For a given degree of saturation, in all cases the modulus of deformation increases with density. The same is true of strength as may be seen from Figures 14b and 14c. It is interesting to compare the curves in Figure 14a with those showing the relationship between density and modulus of deformation at constant values of water content in Figure 9. While the stability of the soil at a given water content may increase or decrease with an increase in density, it may be concluded from Figure 14 that, within the range of densities and water contents investigated, for a given degree of saturation, the stability will increase with an increase in density and, for a given density, the lower the degree of saturation the

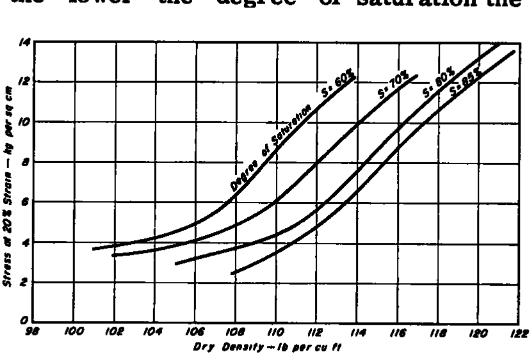


Figure 14b. Relationship between dry density and stress at 20% strain for constant degrees of saturation.

higher will be the stability.

USE OF LINE OF OPTIMUMS FOR PREDICTING STABILITY CHANGES

It has been suggested (7) that the density and water-content conditions at which further increase in density causes a reduction in stability might be predicted from the position of the line of optimums determined by compaction tests. The line of optimums is the smooth curve passing through the peaks of a series of compaction curves and is usually approximately parallel to the zero air-voids curve. Consideration of the fact that the density versus stability relationship is dependent on the definition of stability will show that

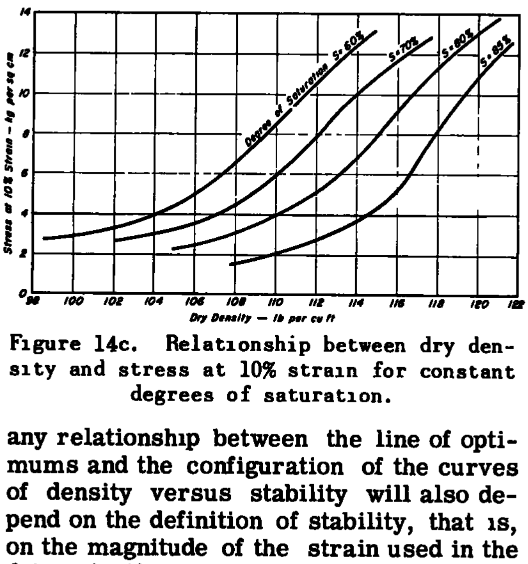


Figure 14c. Relationship between dry density and stress at 10% strain for constant degrees of saturation.

any relationship between the line of optimums and the configuration of the curves of density versus stability will also depend on the definition of stability, that is, on the magnitude of the strain used in the determination of stability. It is somewhat fortuitous, therefore, if the position of the line of optimums, when plotted on the curves of the density versus stability, happens to coincide with the peak points of these curves.

This may be seen from the results in Figures 6 to 11. The line of optimums determined by the compaction curves (Figures 6, 7, and 8) has been plotted on the curves of density versus modulus of deformation in Figure 9 and on the curves of density versus strength in Figures 10 and 11. It will be seen that the peak points of the curves of density versus modulus of deformation occur at densities lower than those determined by the line of optimums; that if strength is determined on the basis of 10 percent strain, the line of optimums

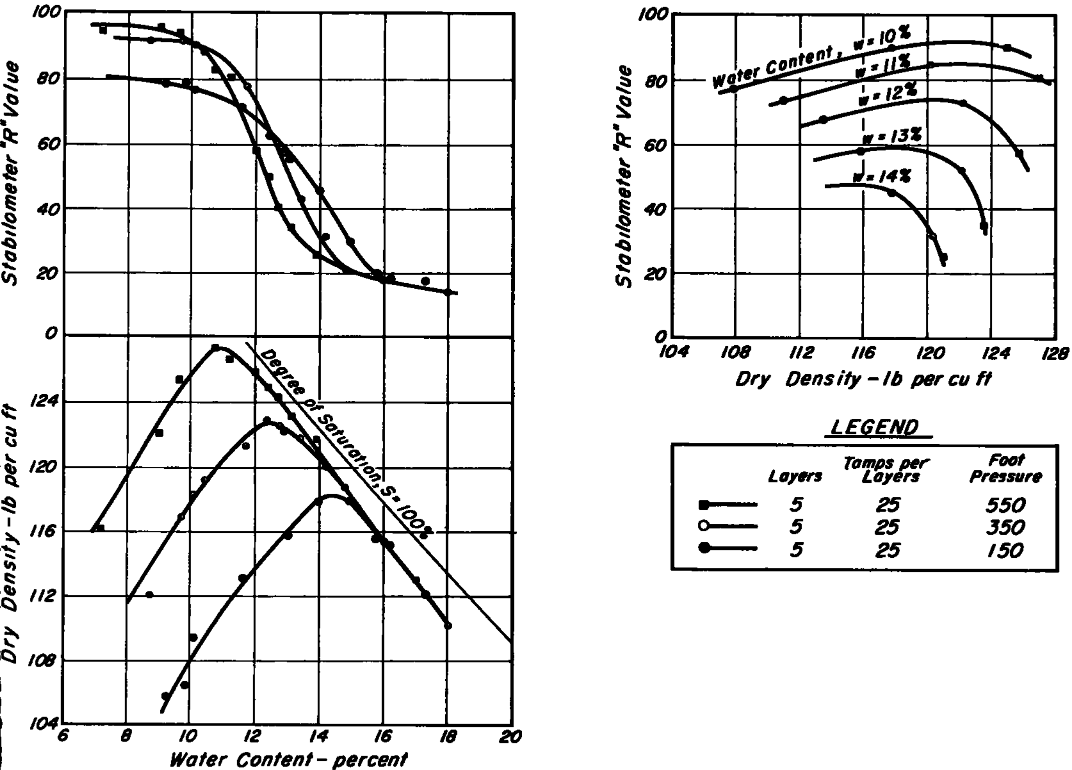


Figure 15. Water content, density, and stability relationships for sandy clay for kneading compaction.

corresponds fairly closely with the peak points of the density-versus-strength curves; and that if strength is determined on the basis of 20 percent strain, the peak points of the curves of density versus strength occur, if at all, at densities considerably higher than those determined by the line of optimums.

Thus it would appear that if stability is defined as the stress required to cause 5 to 10 percent strain in the triaxial-compression tests, the line of optimums will be a good indicator of the density and water content conditions at which further increase in density will cause a reduction in stability; for other amounts of strain, the method would be unsatisfactory. It is interesting to note that the magnitude of the strain for which the line of optimums gives a satisfactory indication of the peak points on the density versus stability curves is of the same order of magnitude as that used in the CBR test. It was for stability determinations made by this test that the use of the line of optimums for indicating peak points of stability was suggested and found to be relatively satisfactory. A similar approximate correlation should not necessarily be expected, however, if other methods of stability determination are used.

EFFECT OF COMPACTION METHOD ON THE STABILITY VERSUS DENSITY RELATIONSHIP

In order to determine whether the general form of the relationship of stability versus density established for the Vicksburg silty clay is applicable to other types of soil and to determine whether the form of the relationship is affected by the method of compaction, two series of tests were made on a sandy clay from Antioch, California.

All samples of soil were mixed and conditioned in the same manner as that used for the silty clay soil previously described.

In the first series of tests, samples 4 inches in diameter and $4\frac{1}{2}$ inches in height were prepared using the kneading compactor. Each sample was compacted in five layers using 25 tamps per layer.

Samples were prepared using tamping pressures of 150, 350, and 550 psi. After the density of the compacted soil had been determined, the upper 2¼ inches of the sample was trimmed off and used for water content determination while the lower 2¼ inches was subjected to a Hveem Stabilometer test and a measure of its relative stability obtained by determination of the resistance value. The results of these tests are shown in Figure 15. It will be seen that there is the characteristic overlapping of the curves of stability versus water content for the various compactive efforts and that the general form of the curves of stability versus density at constant water contents is similar to that obtained for the silty clay soil.

In the second series of tests, samples 4 inches in diameter and about 2¼ inches high were prepared by subjecting the soil to static pressures of 250, 500, and 1,000 psi. These pressures were chosen to give a range of densities approximately the same as that obtained in the kneading-compaction tests. The samples were compacted in metal molds with plungers top and bottom and the pressure was increased to the desired value at a loading rate of 600 lb. per min. After compaction the resistance value of each specimen was determined by a Hveem Stabilometer test. The results are shown in Figure 16.

In this case there is no overlapping of the curves for stability versus water content for constant compactive efforts, and the curves of stability versus density for constant water contents show a consistent increase in stability with increasing density. Thus for the same soil, the general form of the relationship of density versus stability for specimens prepared by static compaction is quite different from that for specimens prepared by kneading compaction. Conclusions with regard to the density-stability relationship drawn from tests on statically compacted soils may therefore be entirely erroneous when applied to soils compacted by kneading action.

In the majority of pavement-design procedures, the samples on which stability

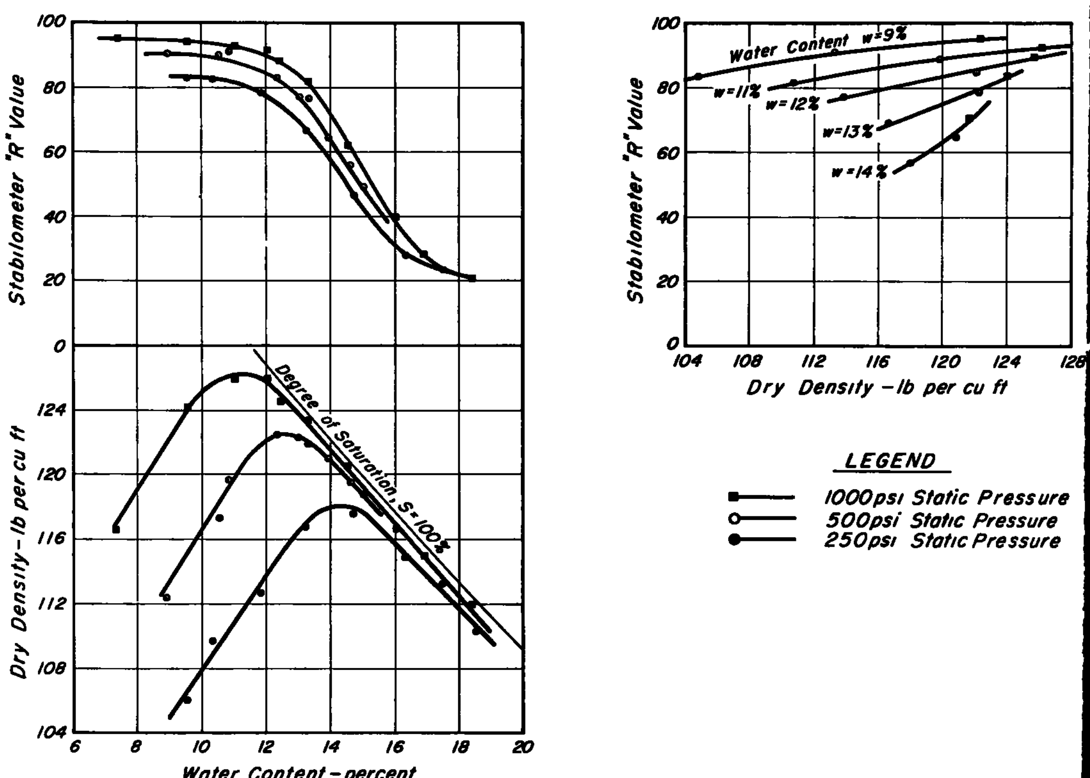


Figure 16. Water content, density, and stability relationships for sandy clay for static compaction.

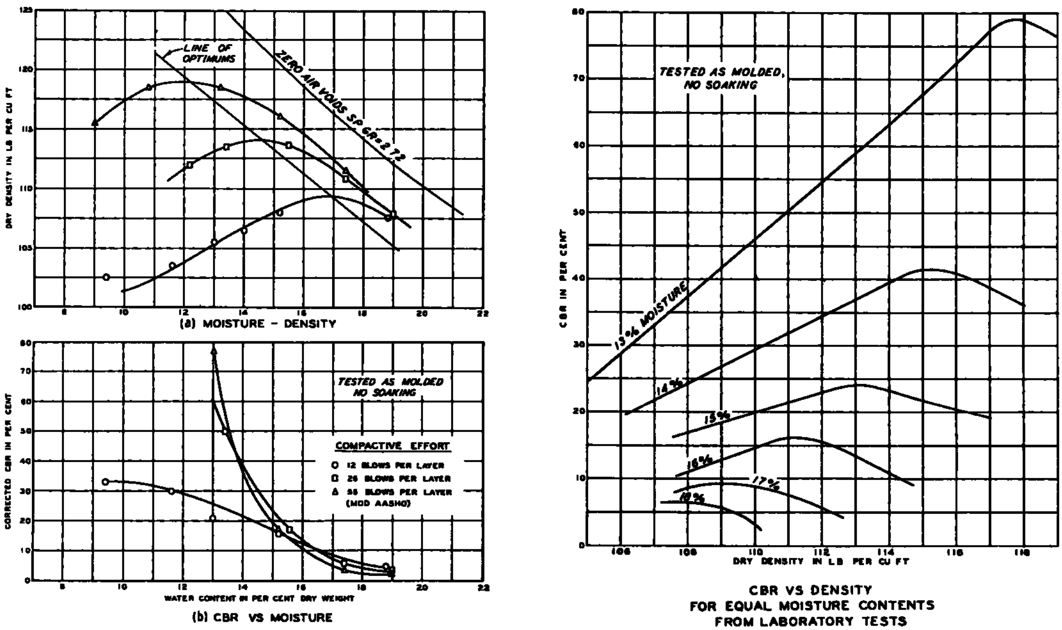


Figure 17. Water content, density, and stability relationships for sandy clay for impact compaction (courtesy C.R. Foster).

tests are made are prepared by impact compaction. The question is thus raised as to how closely this method of compaction will reproduce the properties of soils compacted in the field. At the present time only limited information is available on this aspect of soil compaction, due to the difficulty of obtaining data for field-compacted soils. However, considerable data

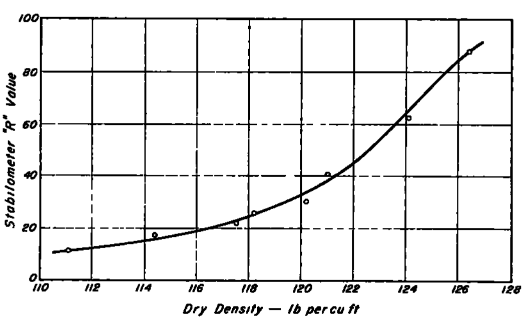


Figure 18. Relationship between density and stability for saturated samples of clayey sand for kneading compaction.

has been obtained by the Corps of Engineers on the density-versus-stability relationship of soil samples prepared by impact compaction. Typical results obtained by the Corps of Engineers in tests of this type, in which the California Bearing Ratio is used as an index of soil stability, are shown in Figure 17. The general form of the curves is similar to that obtained for the specimens prepared by kneading compaction. Although samples prepared by impact compaction show some divergence from the properties of field-compacted soils (3), they have the same general characteristics as samples prepared by kneading compaction and show the same loss of stability, in spite of increased density, at the higher degrees of saturation.

DISCUSSION OF RESULTS

The primary purpose of this investigation was to determine the limitations of the widespread belief that soil stability increases with increasing density. The results have shown that for soils compacted by impact methods or kneading action, the validity of this concept depends on the criterion of stability adopted; when larger strains are permissible in defining a stable soil mass, the concept may be quite true. But for low strains and strains of the order used in the CBR and Hveem Stabilometer tests, an increase in density at a given water content may cause a decrease in stability depending on the range of densities and the water content of the soil involved. For samples of a soil compacted by static forces, however, an increase in density was always associated with an increase in stability as measured by a Hveem Stabilometer.

While it is believed that kneading compaction most-closely duplicates the action of rollers in the field and that compacted subgrades may become less stable if compacted beyond a certain limit, this result does not necessarily mean that every effort should not be made to obtain the highest possible density. The tests reported in this investigation show that for a given degree of saturation, stability always increases with increasing density. This result is apparently equally true for all degrees of saturation, including the condition of complete saturation. Typical results of the relationship between density and stability for saturated soil samples are shown in Figure 18. Thus, if a subgrade is likely to become saturated at any time in the life of the pavement which it supports, the higher the density to which it can be compacted without subsequent swelling, the higher will be the subgrade stability for which the pavement can be designed. Since the large majority of pavements are designed for the saturated condition, compaction to the highest practical density is justified.

It is only for partially saturated soils that increased density may have a deleterious effect on stability. It is necessary, therefore, to take this effect into consideration when the most-economical pavements are to be designed for areas in which it is reasonably certain that the supporting soil will not become saturated. The importance of the relationship between density and stability under those conditions has been recognized for some years by the Corps of Engineers and has been incorporated into their pavement-design method (8).

The density-versus-stability relationship is also important in the design of base courses protected by asphaltic membranes, such as those used in the construction of the fill sections of the Houston Freeway and in other areas (11). By this means, changes in water content of a soil may be prevented and soils which would have low stability when saturated may be safely used for base-course construction. By selecting the water content at compaction and the desirable density in accordance with the relationship of density versus stability, such soils may develop extremely high stabilities if properly protected. However, the desirable water content and density for this purpose would have no particular relationship to the optimum water content and maximum density determined by a standard compaction test.

Finally, it would seem desirable to mention briefly the difficulty of determining accurately the stability of specimens having water contents varying along their lengths. The significance of this effect was clearly demonstrated in the triaxial compression tests on the silty clay, when on one occasion, the porous base plate of the specimen was accidentally wetted and after placement of the test specimen, the lower $\frac{1}{10}$ inch or so of the 4-inch-tall specimen absorbed this moisture. Instead of the anticipated modulus for 1 percent strain of about 800 kg. per sq. cm., the test data showed the specimen to have a modulus of about 520 kg. per sq. cm. This low value could not be accounted for until the water in the bottom of the specimen was discovered during the dismantling of the apparatus. Assuming a modulus of 40 kg. per sq. cm. for the bottom $\frac{1}{10}$ inch of the specimen, it may readily be shown that although the remaining 3.9 inches of the specimen had a modulus of about 800 kg. per sq. cm., the overall modulus of deformation would appear to be about 520 kg. per sq. cm. Such results emphasize the need for careful testing techniques and procedures and the obvious difficulty of interpreting the stability of nonuniform test specimens.

CONCLUSIONS

The main conclusions presented in the preceding pages may be summarized as follows:

1. The relationship between density and stability of soil depends on the criterion used to define stability: the greater the permissible strain before a sample is considered unstable, the greater is the possibility that an increase in density will cause an increase in stability.

2. For samples of two soils, a silty clay and a sandy clay, compacted by kneading action, an increase in density at a given water content caused an increase or a decrease in stability (for strains less than 10 percent) depending on the water content and the range of densities in-

volved; however at a constant degree of saturation, an increase in density always caused an increase in stability.

3. The relationship between the line of optimums determined by compaction tests and the density and water content conditions at which further increase in density will cause a reduction in stability will depend on the method used to evaluate stability.

4. Samples prepared by impact compaction have the same general characteristics as samples compacted by kneading action.

5. Samples of a sandy clay compacted by static pressure always showed an increase in stability, as measured by a Hveem Stabilometer, for an increase in density at constant water content, even though samples compacted by kneading action showed no consistent relationship.

6. For saturated subgrade conditions, the higher the density of the subgrade the greater will be its stability. However for partially saturated subgrades, the desirable density for maximum stability will depend on the water content of the subgrade and too high a density may have a deleterious effect on stability.

7. In designing and constructing pavements resting on partially saturated subgrades, the selection of the desirable density of the subgrade should be based on the anticipated maximum water content of the subgrade and the density versus stability relationship for the soil.

ACKNOWLEDGMENT

The assistance of F. N. Finn and C. K. Chan in performing some of the tests described in this paper and of G. Dierking, who prepared the figures, is gratefully acknowledged.

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Discussion

ROBERT HORONJEFF, Lecturer and Research Engineer, Institute of Transportation and Traffic Engineering, University of California — Seed and Monismith have presented an excellent paper on the relationships between density, water content, and stability. Of particular interest to the writer are the relationships between various degrees of saturation on the modulus of deformation of a soil as shown in Figures 14a, 14b, and 14c. As the axial strain is increased the modulus of deformation is affected less by the degree of saturation. For example, for a density of 110 lb. per cu. ft. an increase in degree of saturation from 60 to 85 percent results in a reduction in the modulus of deformation in the amount of 550 kg. per sq. cm. when the strain is one percent, whereas when the strain is 20 percent the reduction in modulus is only 26 kg. per sq. cm.

It is unfortunate that the authors were not able to include the relationships shown in their paper for a strain of 5 percent. It is the writer's opinion that 5 percent strain represents an upper limit of strain for defining stability as the authors have defined it. It would be interesting to compare the relationships for 1 percent strain with the corresponding relationships with 5 percent strain.

The authors have shown conclusively that generally it is most advantageous from a strength standpoint to compact earth fills on the dry side of optimum regardless of the compactive effort.

Most pavement-design procedures require that the thickness of the pavement structure be predicated on a saturated subgrade. Seed and Monismith have stated that their relationships apply to partially saturated soils; also that the Corps of Engineers has given considerable attention to the relationship of density, water content and stability. The writer has taken some data developed by the Corps of Engineers to show the importance of the moulding water content. Figure A shows the relationships of density and water content to strength for a silty clay. The chart on the right-hand side of the figure shows the relationship between

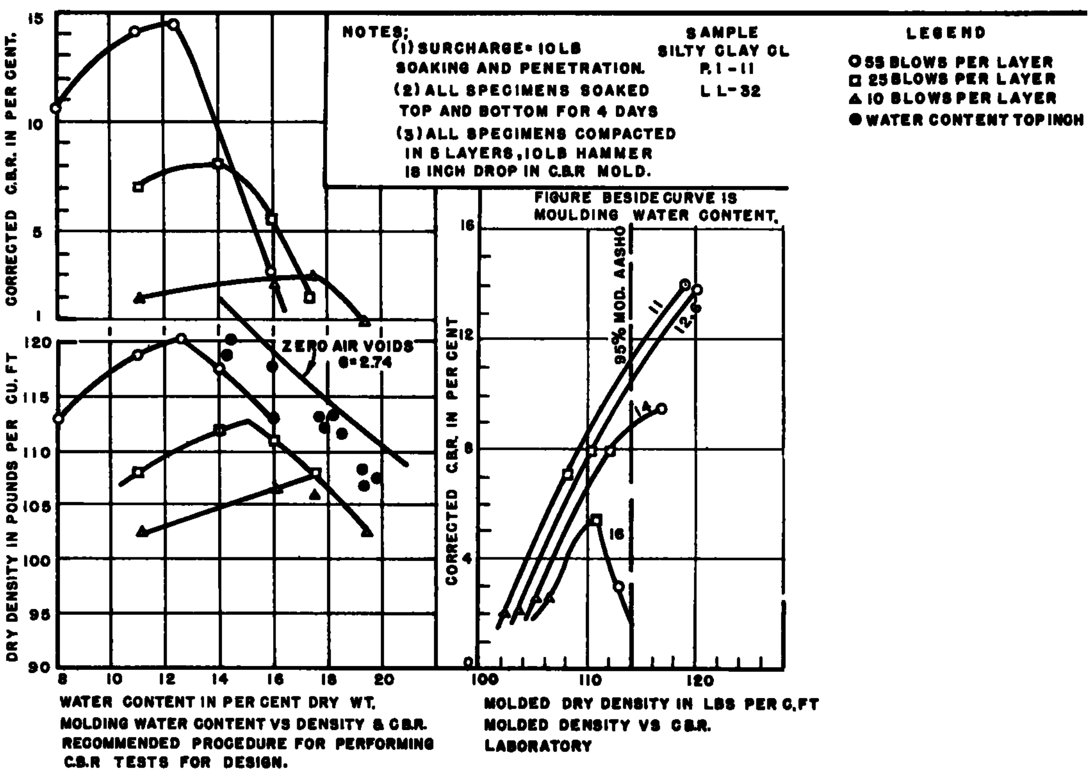

Figure A.

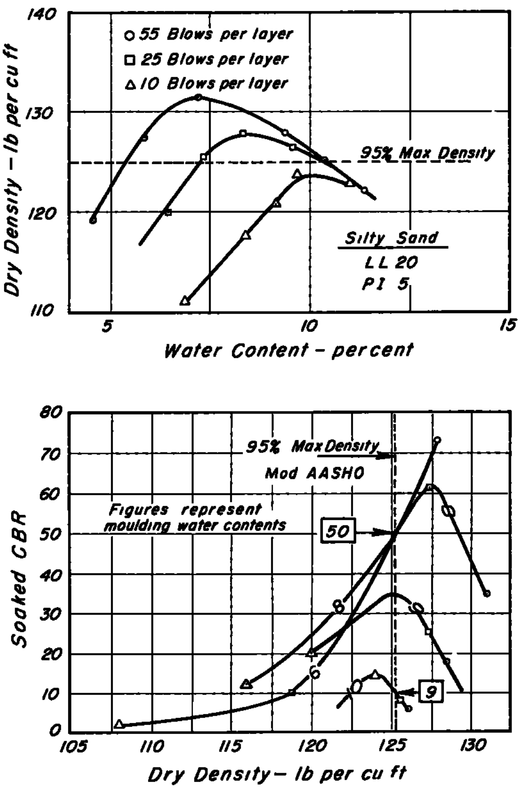

Figure B.

soaked CBR and density for various moulding water contents. The chart is similar to the relationships presented by the authors in Figures 9, 10, and 11, except that the strength is expressed in terms of soaked CBR. The chart supports the authors findings that the greater strengths occur on the dry side of the optimum water content.

The writer recognizes that soaking a soil sample for 4 days does not necessarily mean it is saturated, not even in the top inch or so. Nevertheless there are many soils which are close to full saturation within the depths influenced by the CBR test piston after a 4-day soaking period. Thus one can see that the moudling water content has a profound effect on the strength of a soil in a saturated or nearly saturated condition. From a practical standpoint the importance of controlling the water content in the field is evident. Again referring to Figure A for a specified density of 114 lb. per cu. ft., a range in water contents between 11 and 13 percent does not affect the CBR values very much. However, if the water content were allowed to increase to 16 percent the CBR would be reduced materially. Considering a single wheel load of 50,000 lb. and a tire pressure of 100 psi., an increase in water content from 11 to 16 percent means a reduction in CBR from 11 to 2. This reduction would require an increase in pavement thickness of over 20 inches.

Figure B shows a similar relationship for a sandy clay. It will be noted that for a specified density of 125.5 lb. per cu. ft. an increase in water content from 8 to 10 percent reduces the CBR from 50 to 9.

The effect of moulding water content on the stability of a soil cannot be overemphasized. The moulding water content affects not only the stability immediately upon completion of construction of a pavement but also the stability when the subgrade soil reaches a nearly saturated condition.

W. H. CAMPEN, Manager, Omaha Testing Laboratories — This paper is not only interesting but it also deals with a fundamental aspect of soil stabilization. In fact, the principal purpose of the authors is to show that excess densification of soils is not only possible but that it may result in loss of strength.

Before expressing my views on the main subject of the paper, it should be mentioned that the data in the paper substantiates two well-established properties of soil: (1) density of compacted soils increases with the compactive effort and (2) strength of compacted soils increases as the moisture content decreases with all compactive efforts.

The data in the paper also show that at a given moisture content the strength increases with density up to a point where the compactive effort produces excessive kneading or manipulation. This compactive effort is somewhat greater than the one required to produce optimum moisture equal to the given moisture content. At this point an increase in density takes place but the strength decreases, if strength is measured at strains similar to those obtained in the CBR test.

The increase in density must be due to the expulsion of air or gases. The reduction in stability is no doubt due to an increase in the lubricating action of the clay portion of the soil. The fact that compaction by static force does not reveal this behavior substantiates this assumption.

While it should be recognized that in the laboratory it is possible to reduce strength by increasing density, it must be pointed out the deleterious effect does not have much practical significance. For instance, the data in Figure 17, obtained by the modified AASHO method of compaction and the CBR method of measuring strength, shows that the soil used has a maximum density of 119 lb. and an optimum moisture of 12 percent. This soil might be compacted in the field to a relative density of 95.5 percent or 114 lb. per cu. ft. At this density the soil has an optimum moisture of 14.75 percent and a CBR of 27 percent. At a density of 116 lb. at the same moisture content the soil has a CBR of 25 percent. If the soil were compacted to 99.5 percent or 117.5 lb. per cu. ft., the corresponding optimum moisture would be 13 percent. At this water content and density the soil has a CBR of 79 percent. At a density of 119 lb. and the same water content it has a CBR value of 77 percent. Thus it will be noted that the loss of strength due to over densification is comparatively small.

Even though it can be shown in the laboratory that a loss of strength can accompany an increase density, it is doubtful if the same thing can happen in the field. It may be that compaction by sheepsfoot rollers can produce this result, but it is unlikely that traffic can do so. The rollers might do it if compaction were done at constant moisture content by a series of rollers capable of applying pressure in ascending order. On the other hand, traffic could hardly do it for the reason that it cannot expel air or apply a kneading action. These two reactions can hardly take place, since the subgrade soils are usually covered with a considerable thickness of superimposed layers.

In connection with this discussion, it would seem appropriate to mention loss of stability in bituminous mixtures, because it is often confused with similar occurrence in soils. Traffic can and often does reduce the stability of bituminous mixtures to a large degree. That is because the traffic acts directly on the mixtures, which are usually used as wearing surfaces, and thus densifies them, if improperly designed, to a point where the asphaltic cement becomes a lubricant. On the other hand, it has already been pointed out that soils are not likely to be densified or activated by traffic, and even if they are, the loss of strength is comparatively small.

In concluding the comments, I wish to emphasize the fact that water is the principal foe of stability, and since the amount of water can be limited by density, as much density as possible, within practical limits, should be developed in the compaction of soils.

H. B. SEED and C. L. MONISMITH, Closure — The authors express their appreciation to Horonjeff and Campen for their interesting discussions. In closing, it would seem desirable to comment on Campen's observations that the deleterious effect of too high a density does not have much practical significance and that it is doubtful if the same thing could happen in the field. Campen reaches these conclusions by using data from the curves for samples prepared by impact compaction. The stabilities of samples prepared by this method are not so greatly affected by changes in density as are those of samples prepared by kneading compaction. (Seed, H. B., Lundgren, R., Chan, C. K., "The Effect of Compaction Method on the Stability and Swell Pressure Characteristics of Soils," Proceedings of the Highway Research Board, 1954).

The optimum water contents for samples compacted to a given density by field equipment are somewhat greater than those determined by impact methods, and an analysis based on compaction and stability data for samples prepared by kneading compaction would show that density changes of the order considered by Campen can have a substantial effect on stability values. That this effect can be of practical significance and that the same effect can occur in the field is illustrated by a recent paper (Foster, C. R., "Reduction in Soil Strength with Increase in Density," Proceedings Separate No. 228, New York: American Society of Civil Engineers, July 1953).

Results obtained by the Corps of Engineers and presented in this paper show that a large change in stability occurred in the field as a result of an increase in density under the action of heavy traffic.

Effect of Compaction Method on Stability and Swell Pressure of Soils

H. B. SEED, Assistant Professor of Civil Engineering,
Institute of Transportation and Traffic Engineering, University of California,
RAYMOND LUNDGREN, Partner,
Woodward, Clyde and Associates, Oakland, California, and
CLARENCE K. CHAN, Graduate Research Engineer,
Institute of Transportation and Traffic Engineering, University of California

Test data are presented for two soils, a silty clay and a sandy clay, comparing the stabilities at various water contents and densities of partially saturated samples compacted by impact methods, static pressure, and kneading action; the stabilities of samples prepared by these three methods and soaked to near saturation at constant density are also compared. A possible explanation for the different effects of the compaction methods is suggested and some of the difficulties of preparing saturated samples in the laboratory are discussed.

Data are also presented comparing the deformation characteristics in triaxial-compression tests of silty-clay specimens prepared by impact, static, and kneading compaction in the laboratory with those of the same soil compacted by sheepsfoot and rubber-tired rollers in the field.

A comparison is also made of the stability and swell pressures developed at various densities for samples of a sandy clay compacted by kneading and static methods and subsequently saturated by exudation of moisture under static load; the great difference in test results obtained is illustrated.

● THE object of a laboratory compaction test is to reproduce in the laboratory the compaction effects produced by equipment in the field. Not only should laboratory-compacted samples exhibit the same relationship of density versus water content as the soil compacted in the field, but since the laboratory samples are used for design purposes, they should also possess the same deformation characteristics under load. At the present time, three main methods of compaction are in use for the preparation of samples in the laboratory. In the majority of tests, the soil is compacted by dropping a weight onto the surface of the soil, a process referred to as impact compaction. In some cases the soil is compacted by subjecting it to a static load which is built up slowly to some predetermined value and then released, a process referred to as static compaction. In other methods, the soil is compacted by repeatedly applying a predetermined pressure to small areas of the soil, maintaining the pressure for a small element of time and then gradually reducing the pressure, a process termed kneading compaction.

It has long been recognized that samples having the same water content and density but prepared by impact and static compaction have different stress-strain characteristics (1). Recent investigations have shown this to be true also for samples prepared by kneading and static compaction (2). It becomes important, therefore, in order to satisfactorily design a pavement on the basis of laboratory tests, to know which method of compaction reproduces most closely the effects of field equipment and the magnitude of the differences in stability of a sample resulting from the use of different compaction methods.

Relatively little information is available on the extent to which the stabilities of laboratory compacted samples compare with those of samples compacted in the field. Some tests conducted by the Corps of Engineers have shown that samples of a silty clay taken from the field have different stress-strain characteristics from those prepared in the laboratory by impact and static compaction; additional data are presented in this paper to compare these results with those for the same soil compacted by kneading action. However, the main purpose of the investigations described is to illustrate the extent to which different methods of laboratory compaction affect the stability of soils and to draw tentative conclusions with regard to the qualitative nature of these effects.

In the pavement-design procedure used by the California Division of Highways, the expansion or swell pressure developed by a soil is used to determine the desirable pavement thickness. The effect of compaction method on the swell-pressure characteristics of soils, as measured by the California design procedure, is therefore also important and has been included within the scope of the investigation.

density, water content, and stability. For each soil and for each method of compaction, series of tests were made to establish the relationships between dry density and water content and between stability and water content at each of three different compactive efforts. For each soil, the compactive efforts were selected to give results over approximately the same range of densities and water contents for each of the three methods of compaction; this range was approximately between the densities obtained in the standard Proctor and the modified AASHO compaction tests.

For tests at any one compactive effort,

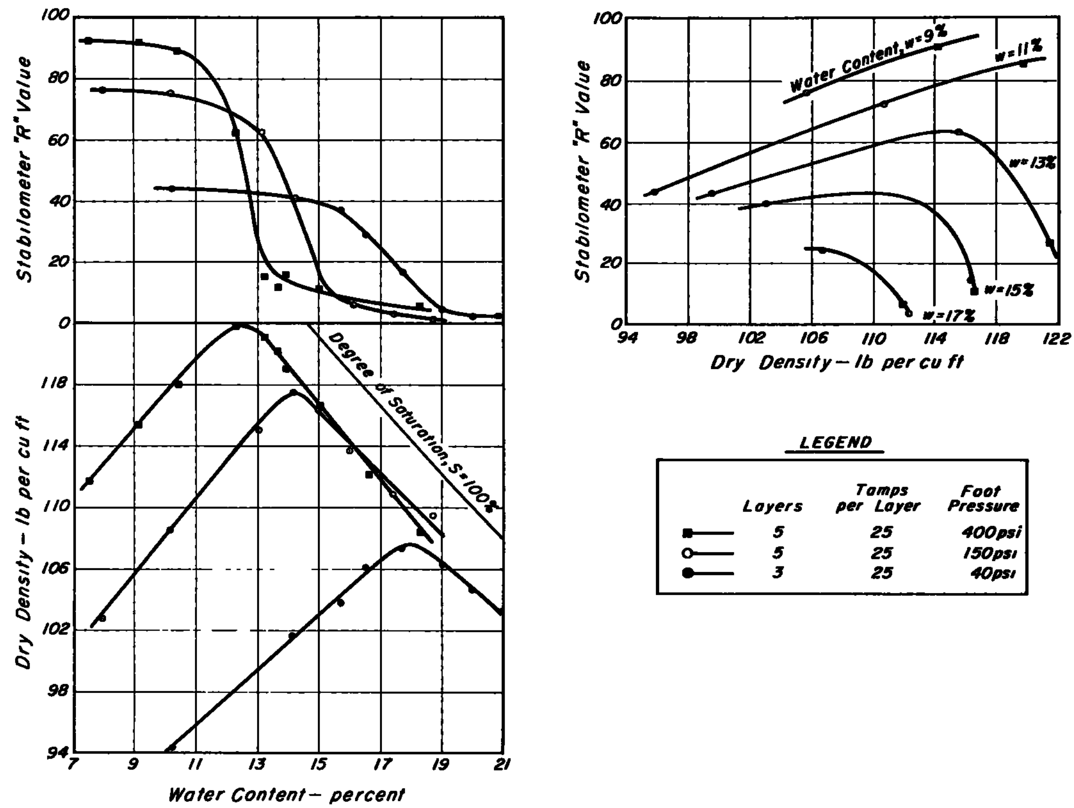

Figure 1. Water content, density, and stability relationships of silty clay for kneading compaction.

EFFECT OF COMPACTION METHOD ON THE STABILITY OF PARTIALLY SATURATED SOILS

Comprehensive series of tests were made on two soils, a silty clay from Vicksburg, Mississippi, and a sandy clay from Pittsburg, California, to determine the effect of impact, static, and kneading compaction on the relationship between dry

the soil was first oven-dried and samples were then mixed at about six different water contents. Each sample was placed in a sealed container and allowed to condition for one day prior to compaction. Specimens 4 inches in diameter and 4½ inches in height were then prepared using the selected method of compaction and compactive effort. After the density of the compacted soil had been determined, the upper 2¼ inches of the specimen was trimmed off and used for water-content determination while the lower $2\frac{1}{4}$ inches was tested in a Hveem Stabilometer and a measure of its stability

the two soils are reproduced in Figures 4 and 8. Figure 4a compares the stabilities at various water contents and densities, for specimens of the silty clay compacted

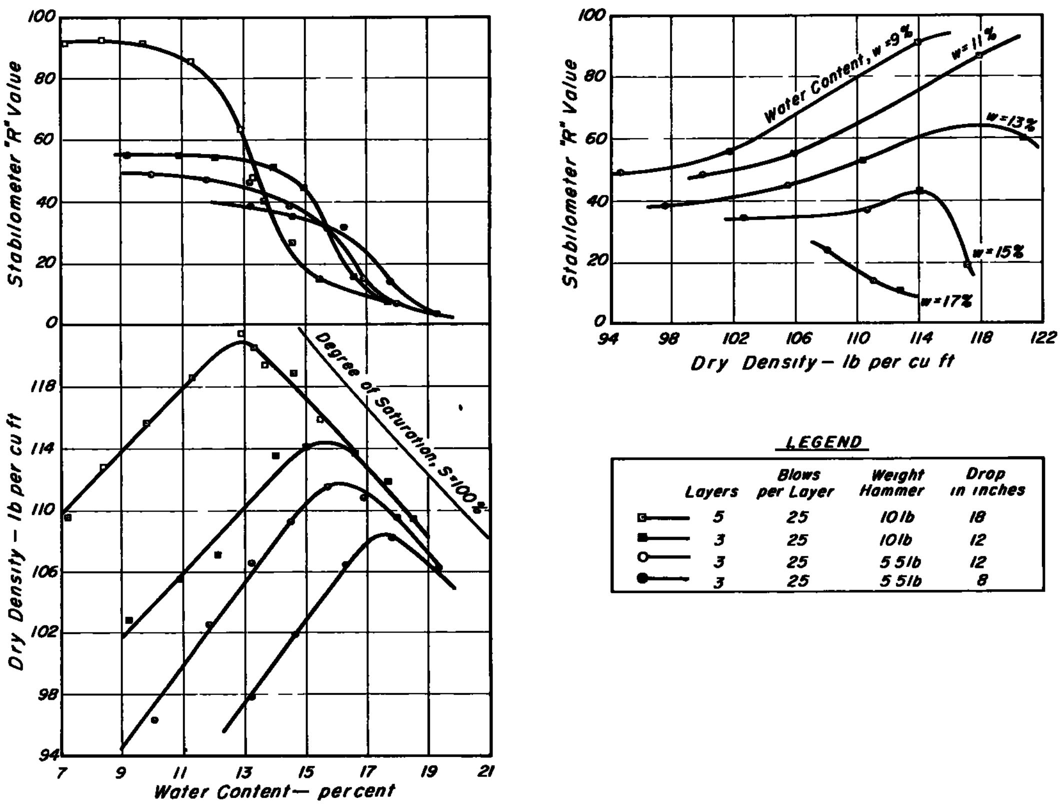

Figure 2. Water content, density, and stability relationships of silty clay for impact compaction.

obtained by determination of the resistance or R value (2). The resistance value is used as an index of stability in the California method and has been correlated with the required thickness of pavement for various types of loading conditions.

The results of these tests on the silty clay, for kneading, impact and static compaction procedures respectively, are shown in Figures 1, 2, and 3. Similar results for the sandy clay are shown in Figures 5, 6, and 7. On the left of each of these figures the test data is presented and on the right is shown the relationship between density and stability at various constant values of the water content; this latter relationship was in all cases interpolated from the test results shown on the left of the figure.

For purposes of comparison, the relationship of density versus stability for by kneading and impact methods. It will be seen that, in general, the curves for these two methods of compaction are similar in form but that kneading compaction produces slightly higher stabilities at the lower densities and impact compaction results in somewhat higher stabilities at the higher densities. The higher densities on the curves in this type of plot are associated with the higher degrees of saturation; thus at higher degrees of saturation impact compaction produces the higher stabilities, while at lower degrees of saturation, kneading compaction produces higher stabilities. Comparison of the densities at which impact compaction begins to give higher stabilities with the position of the line of optimums for the compaction curves in Figures 1 and 2 will show that it is at water contents just below and above the optimum water content for the particular compactive effort being

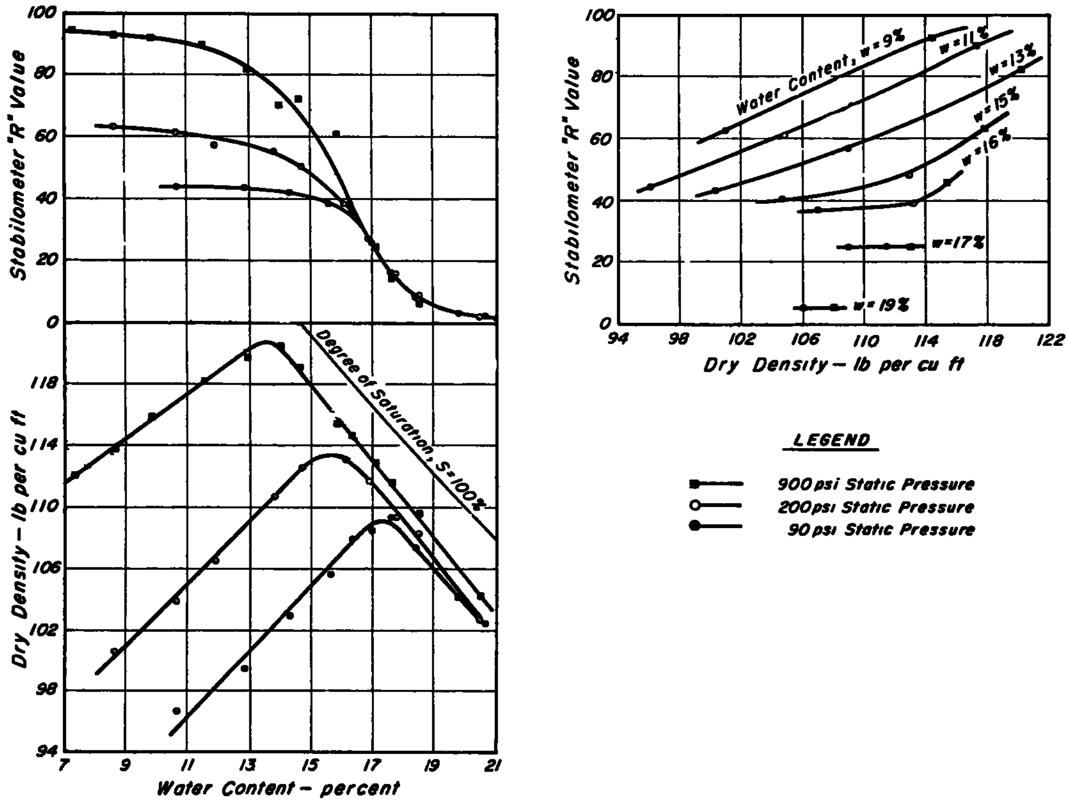

Figure 3. Water content, density, and stability relationships of silty clay for static compaction.

used that impact compaction causes higher stability than kneading compaction; at water contents well below optimum, kneading compaction results in the higher stabilities. However, at no stage is there any great difference between the results obtained by these two methods.

It is interesting to note that both kneading and impact compaction result in samples which, at lower degrees of saturation, show an increase in stability with increase in density at a given water content but at the

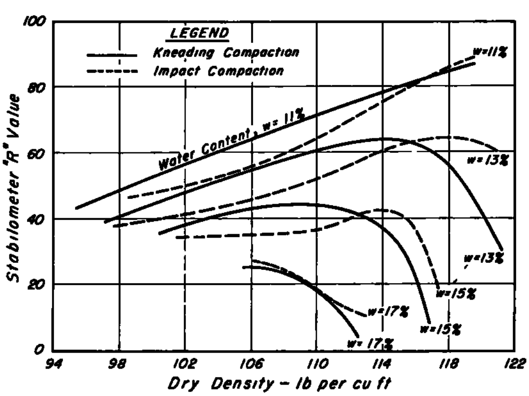

Figure 4a. Effect of kneading and impact compaction on density versus stability relationship at constant water contents for silty clay.

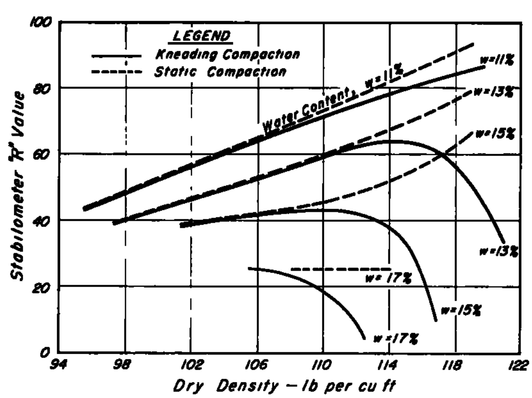

Figure 4b. Effect of kneading and static compaction on density versus stability relationship at constant water contents for silty clay.

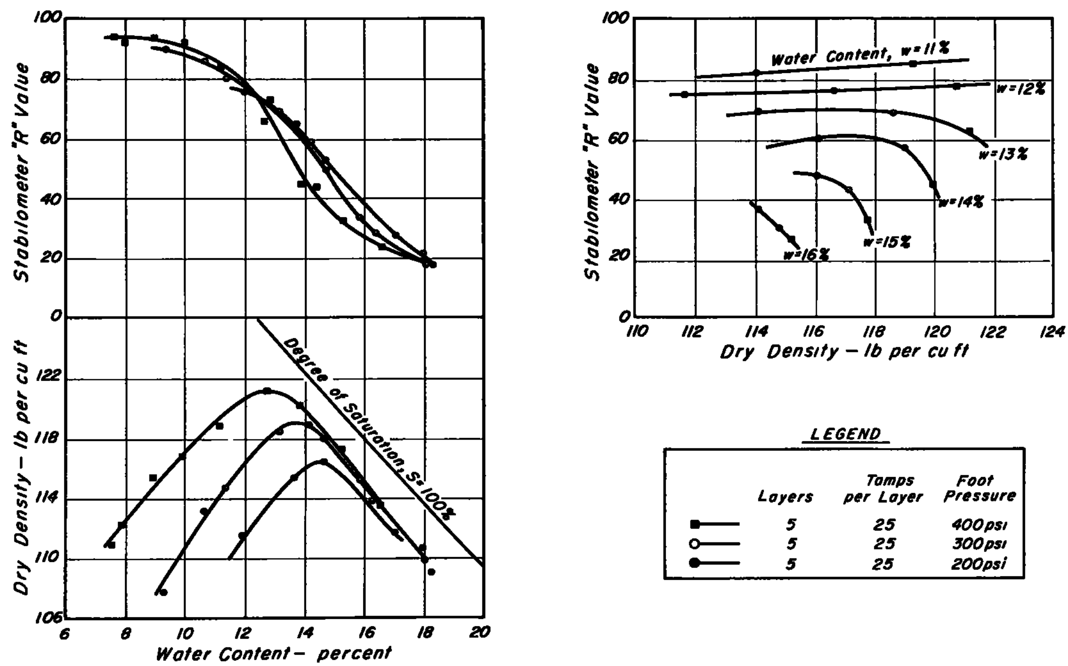

Figure 5. Water content, density, and stability relationships of sandy clay for kneading compaction.

higher degrees of saturation show a reduction in stability with increase in density at constant water content. This reduction in stability with increase in density is more pronounced for specimens prepared by kneading compaction than for those prepared by impact compaction.

The stabilities at various water contents

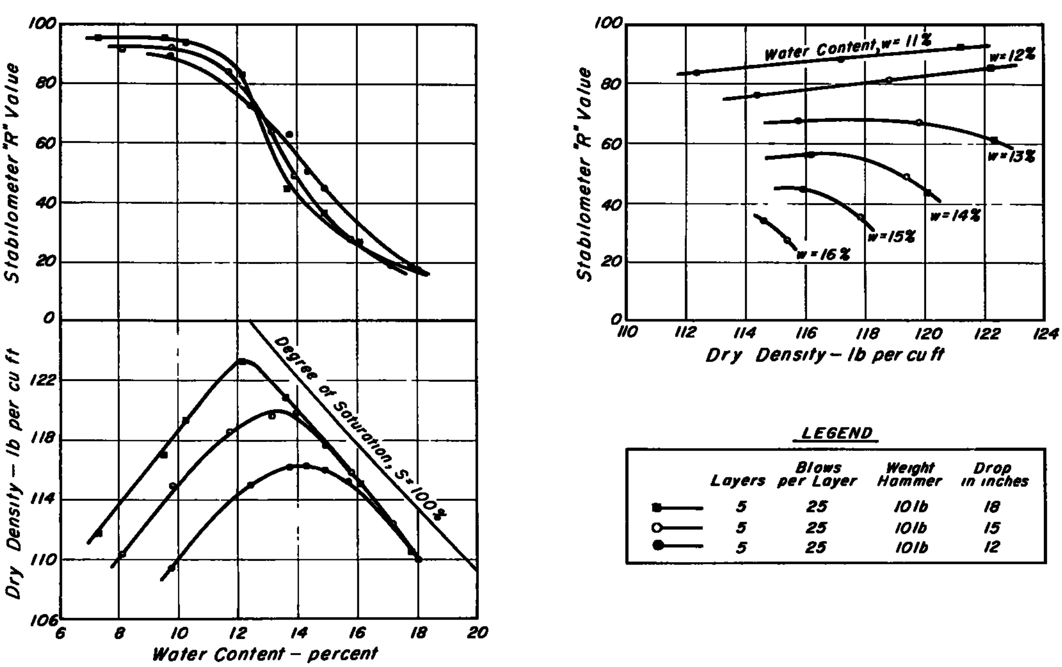

Figure 6. Water content, density, and stability relationships of sandy clay for impact compaction.

and densities of samples of silty clay prepared by kneading and static compaction are compared in Figure 4b. The most-important difference in the results for these compaction methods is that samples prepared by static compaction always show an increase in stability with an increase in density at any water content, while for samples prepared by kneading compaction the stability is reduced if, at water contents greater than about 12 percent, the density is increased at the higher degrees of saturation. This difference results in a considerable discrepancy between the stabilities of samples prepared by static and kneading methods. For example, at a water content of 15 percent and a density of 116 lb. per cu. ft. the resistance value of a sample prepared by kneading compaction is only 22, while that for a sample prepared by static compaction is 56, an increase of approximately 150 percent. In terms of

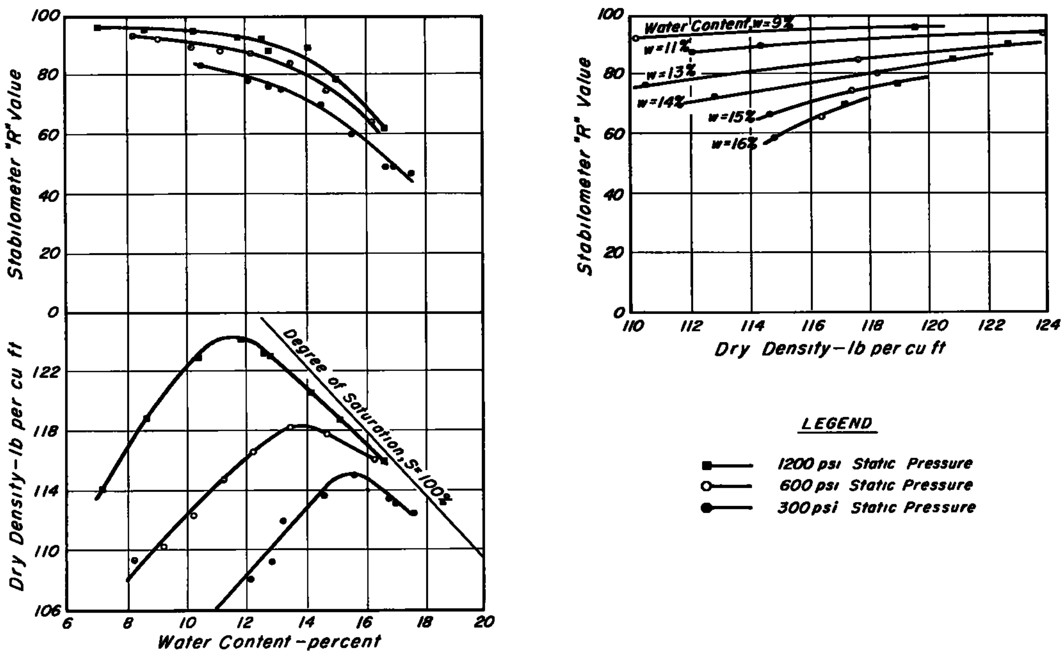

Figure 7. Water content, density, and stability relationships of sandy clay for static compaction.

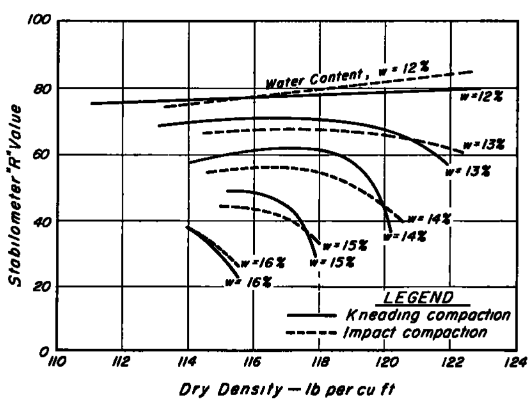

Figure 8a. Effect of kneading and impact compaction on density versus stability relationship at constant water contents for sandy clay.

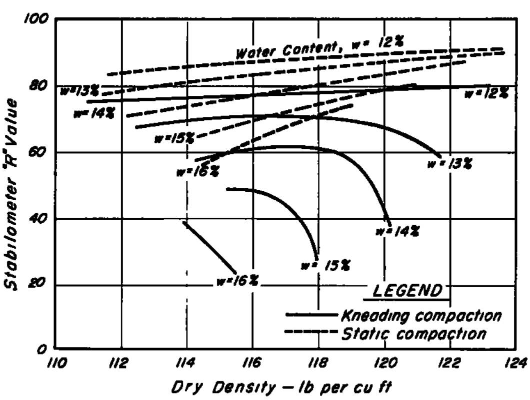

Figure 8b. Effect of kneading and static compaction on density versus stability relationship at constant water contents for sandy clay.

pavement thickness for a typical flexible pavement designed by the California procedure for a highway with heavy traffic, a resistance value of 22 would indicate a required thickness of pavement and base of about 18 inches, while a resistance value of 56 would indicate a required thickness of only about 9 inches. If kneading compaction most satisfactorily reproduces the effects of field compaction, the dangers of designing a pavement for a partially saturated subgrade condition on the basis of tests on samples prepared by static compaction are immediately evident.

At lower degrees of saturation, the stabilities of samples of the silty clay prepared by static and kneading compaction appear to be almost identical. Thus, at equal water contents and densities, the stabilities of samples prepared by static compaction were always equal to or greater than those of samples prepared by kneading compaction. Comparison of the curves in Figures 4a and 4b show this to be true also for static and impact compaction.

The stabilities of samples of sandy clay prepared by kneading and impact compaction are compared in Figure 8a. As for the silty clay, kneading compaction gives slightly higher stabilities at lower degrees of saturation, and impact compaction gives slightly higher stabilities at the higher degrees of saturation; for this soil it is approximately for samples compacted on the dry side of the optimum water content for the particular compactive effort being used that kneading compaction gives the higher stabilities and for samples on the wet side of optimum that impact compaction gives the higher stabilities. However, both methods of compaction again show that at higher degrees of saturation, an increase in density may lead to a decrease in stability.

Comparison of the stabilities resulting from kneading and static compaction of the sandy clay in Figure 8b, shows that, for any given water content and density condition within the range investigated, static compaction gives the higher stability. Furthermore, in contrast to the results for kneading and impact compaction, the samples prepared by static compaction show a consistent increase in stability with increase in density, even at higher degrees of saturation. As a consequence of this, there are again large differences in stability between samples prepared by static and kneading compaction or by static and impact compaction at the higher degrees of saturation. For example, at a water content of 15 percent and a dry density of 118, the resistance value of a sample compacted by static pressure was 75, compared with a resistance value of 27 for a sample prepared by kneading compaction.

From the results presented in Figures 4 and 8 for the two soils investigated, the following general conclusions may be drawn:

1. At a given density and water content, samples compacted by static pressure have higher stabilities than samples prepared by kneading or impact compaction; this is particularly true at higher degrees of saturation when there is a large difference between the stabilities of samples prepared by static and impact or static and kneading compaction.

2. At any given density and water content, samples prepared by impact and kneading compaction have similar stabilities, with kneading compaction resulting in somewhat higher stabilities for samples compacted on the dry side of the optimum water content and impact compaction producing higher stabilities for samples compacted at water contents above the optimum for the particular compactive effort being used.

3. For samples prepared by impact or kneading compaction, an increase in density may cause an increase or decrease in stability depending on the water content and the range of densities involved; for samples prepared by static compaction, an increase in density always results in an increase in stability.

EFFECT OF TAMPING PRESSURE ON STABILITY OF PARTIALLY SATURATED SOILS

In the tests on samples of silty clay prepared by kneading compaction, the different compactive efforts were obtained by varying the tamping pressure from 40 to 400 psi. It is of interest to determine how, for any given water content, the stability of a sample will vary with the tamping pressure used to compact it. Such results may be interpolated from the data in Figure 1, and the variation of the resistance values of samples with the tamping pressure used in the compaction tests, for a series of constant water contents, are shown in Figure 9. The relationships of water con-

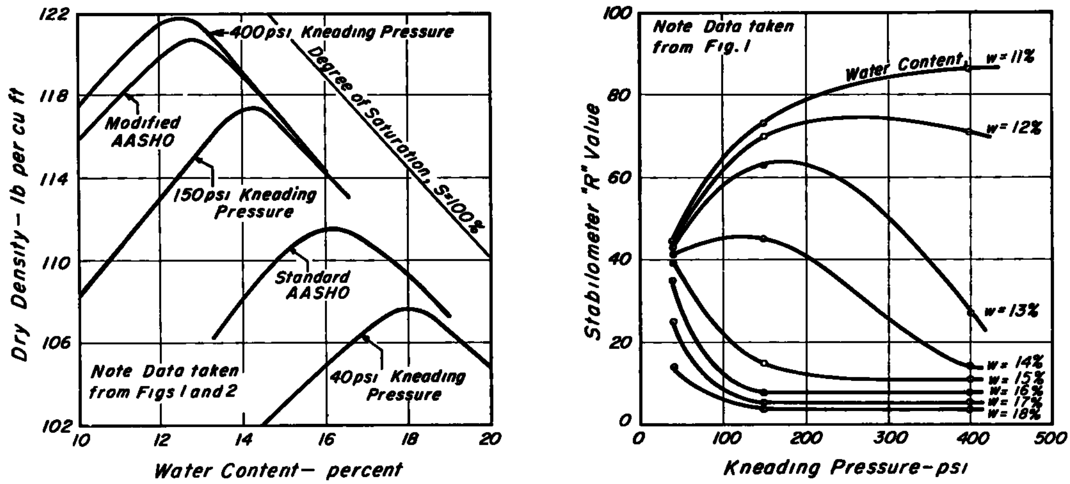

Figure 9. Density versus water content and stability versus kneading-pressure relationships for silty clay.

tent versus density resulting from these tamping pressures are shown on the left of Figure 9, together with the curves obtained by the standard AASHO and modified AASHO compaction tests for comparison.

It will be seen that an increase in tamping pressure may lead to an increase or decrease in stability of the compacted soil depending on the water content at which the soil is compacted. The optimum water content as determined by the modified AASHO compaction test for this soil is about 13 percent. A tamping pressure of 400 psi. results in densities comparable to those obtained by the modified AASHO test; yet, if this pressure is used at a water content of 13 percent, the compacted soil has a lower stability than would be obtained for any tamping pressure between 40 and 400 psi. It is interesting to note that, for compaction at this water content, the maximum stability would be obtained by using a tamping pressure of about 175 psi. which, according to the positions of the compaction curves shown in Figure 9, would produce a relative compaction of about 95 percent. However, an increase in tamping pressure from 175 to 400 psi., would reduce the resulting stability of the soil by over 50 percent.

For compaction at the optimum water content as determined by the standard AASHO compaction test, the stability decreases as the tamping pressure increases

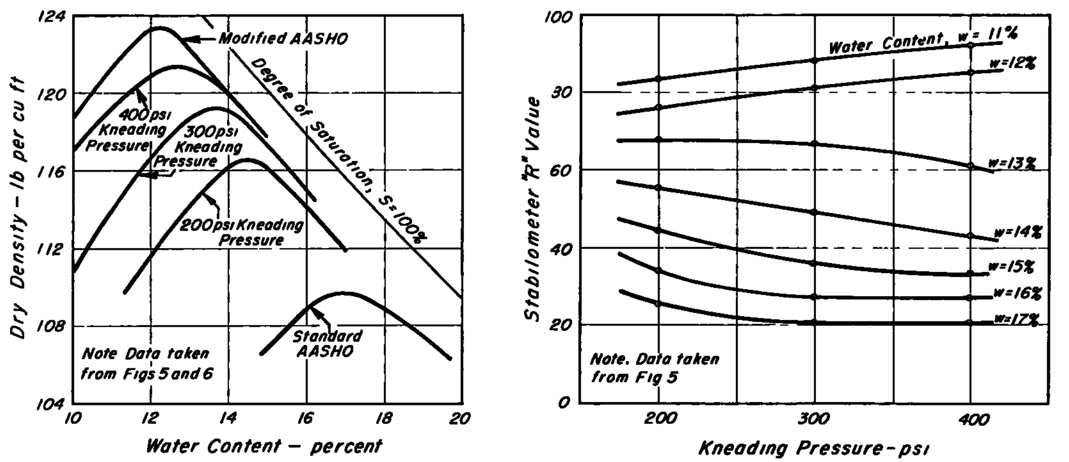

Figure 10. Density versus water content and stability versus kneading-pressure relationships for sandy clay.

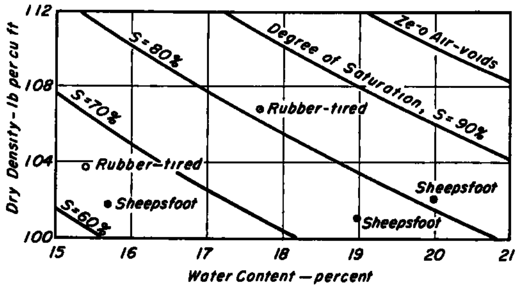

Figure 11. Water content versus density relationship for field-compacted samples of silty clay.

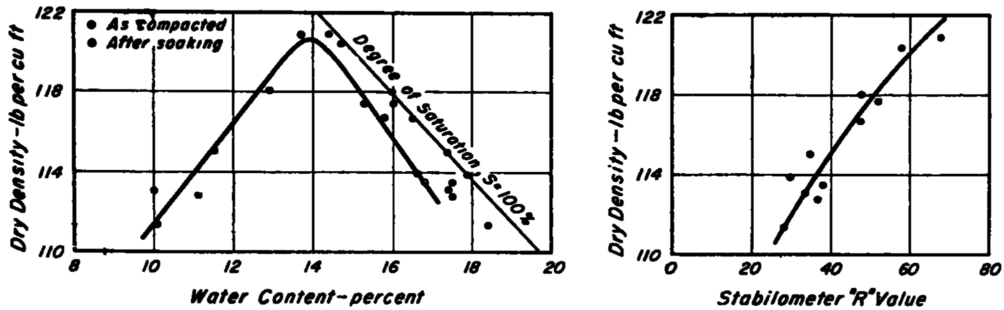

Figure 12a. Density versus water content and density versus stability of samples soaked at constant density for static compaction.

from 40 to 150 psi., but beyond this point, further increase in tamping pressure has no effect on stability; presumably the ultimate bearing capacity of the soil is reached at a pressure of about 150 psi. and tamping pressures exceeding this value cause shear failure and no change in condition of the soil.

The significant effects which the water content and the kneading pressure used for compaction may have on the resulting stability of a partially saturated soil are clearly evident from the curves in Figure 9. For this silty clay at water contents more than 1 percent below the optimum for the modified AASHO compaction test, an increase in tamping pressure at least up to 400 psi. had a beneficial effect on stability; at water contents more than 1 percent above this optimum, an increase in tamping pressure above 40 psi. had a deleterious effect on stability. Near the optimum water content for the modified AASHO test, the most-desirable tamping pressure decreased as the water content increased. Unfortunately, the tamping pressures used in the laboratory tests have not been correlated with those producing similar degrees of compaction in the field, but the nature of the effects produced by field equipment will be similar to those obtained in the laboratory.

The relationships between stability and tamping pressure at various constant values

TABLE 1- SUMMARY OF TEST RESULTS

Water Content - percent	15.4	15.7	17 7	19.0	20.0
Dry Density - lb per cu ft	103.7	101 7	106.8	101.0	102 0
Modulus of Deformation at 1% Strain					
Field Compaction - sheepsfoot roller		320		130	120
Field Compaction - rubber-tired rolle:	400		180		
Lab. Compaction - Impact - Corps of .ng.	330	330	120	120	45
Lab. Compaction - Impact - Univ. of Calif.	290	250	155	120	90
Lab. Compaction - Kneading - Univ. of Calif.	350	300	195	130	90
Lab Compaction - Static - Corps of Eng.	330	320	225	160	-
Percent Difference between Moduli of Deformation for Field and Laboratory Compaction					
Laboratory Impact Compaction	-22.5	-9.5	-23.5	-7.5	-44.0
Laboratory Kneading Compaction	-12.5	-6.5	+8.5	0	-25.0
Laboratory Static Compaction	-17.5	0	+25.0	+23.0	-

of water content for the sandy clay are shown in Figure 10. The general form of these curves is similar to that for the silty clay, though the effect of tamping pressure on stability is not so great. Furthermore, at the optimum water content for the modified AASHO compaction test, an increase in tamping pressure above 200 psi. caused a slight reduction in stability. The effect of water content at compaction, for any given tamping pressure again has an important effect on stability. For a tamping pressure of 300 psi., compaction at the optimum water content for the standard AASHO test (17 percent) would produce a sample with a resistance value of only 21, compared with a value of about 80 for compaction at the optimum water content for the modified AASHO test (12 percent).

of silty clay compacted by sheepsfoot and rubber-tired rollers in the field were compared with those of samples, at the same densities and water contents, prepared by static and impact compaction in the laboratory. Through the courtesy of the Waterways Experiment Station at Vicksburg in supplying some of the soil used in the tests, this comparison has been extended to include samples prepared by kneading compaction.

In the field the soil was compacted in 6-inch lifts by six passes of either a sheepsfoot roller with a foot pressure of 500 psi. or a rubber-tired roller with a wheel load of 20,000 lb. Undisturbed samples were cut from a depth of 12 to 21 inches below the top of the compacted soil and trimmed to 1.4-inch-diameter specimens for test-

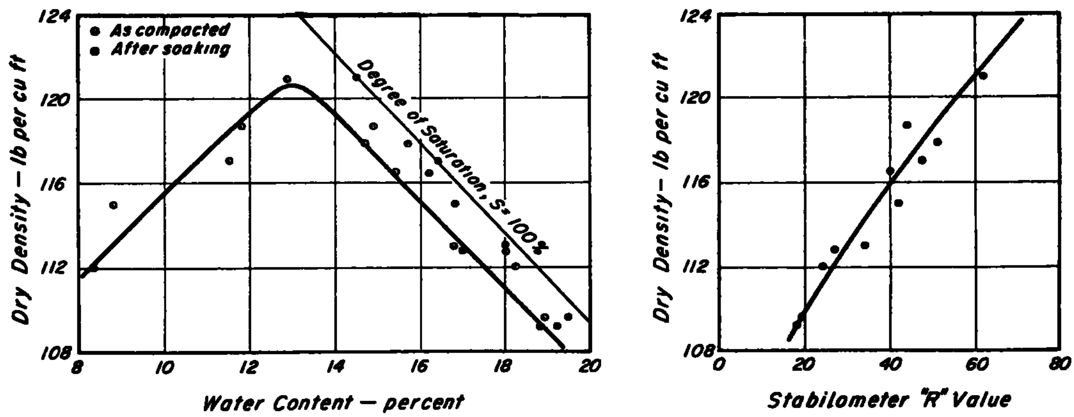

Figure 12b. Density versus water content and density versus stability of samples soaked at constant density for impact compaction.

These results emphasize the importance of careful control of construction conditions and the possible dangers of the arbitrary selection of these conditions on the basis of standard compaction tests, when placing soils for maximum stability under conditions where the water content is unlikely to change appreciably from that at which the soil is compacted.

COMPARISON OF STABILITIES PRODUCED BY FIELD AND LABORATORY COMPACTION PROCEDURES

In a previous investigation conducted by the Corps of Engineers (1), the strength and deformation characteristics, as measured by triaxial-compression tests of the unconsolidated, undrained type, on samples

ing. The water contents and densities of five samples taken from the field are shown in Figure 11, and their moduli of deformation at 1 percent strain, as measured by triaxial-compression tests using a constant lateral pressure of 1 kg. per sq. cm., are presented in Table 1.

In the tests conducted by the Corps of Engineers, the greatest discrepancy between the stress-deformation characteristics of samples prepared in the laboratory by impact and static compaction was found to occur at low strains. In this investigation, the modulus of deformation at 1 percent strain was therefore selected as the basis for comparison of the effects of different compaction methods.

In Table 1 are summarized the moduli of deformation at 1 percent strain of sam-

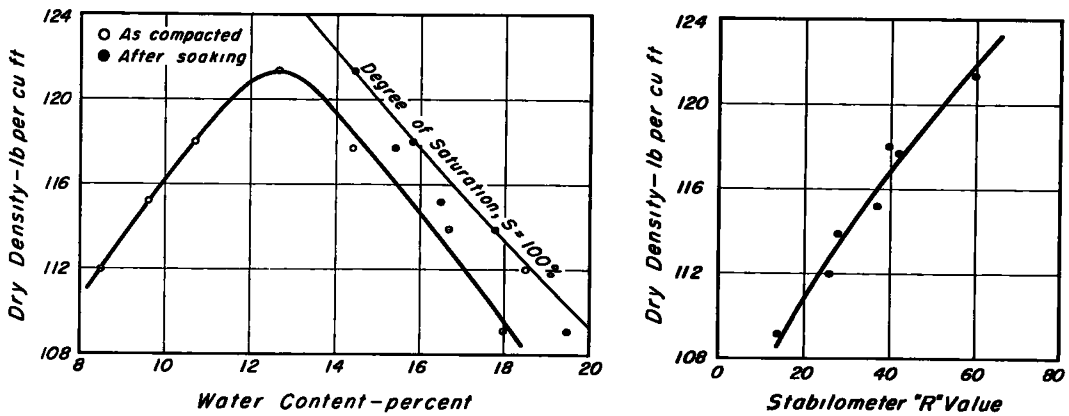

Figure 12c. Density versus water-content and density versus stability of samples soaked at constant density for kneading compaction.

ples at the same water contents and densities as those taken from the field and tested in the same manner but prepared in the laboratory by impact, kneading, and static compaction. In order to ensure that the testing procedure used for the samples prepared by kneading compaction was the same as that used by the Corps of Engineers, test data was also obtained for samples prepared by impact compaction. It will be seen that there is reasonably good agreement between the results of tests on samples prepared by impact compaction conducted in the different laboratories.

To facilitate comparison of the test results, the differences of the moduli of deformation of laboratory compacted samples from those of the field compacted samples, expressed in percent of the moduli of the field compacted samples, are summarized at the foot of Table 1. For samples prepared by impact compaction, the average of the values obtained by the Corps of Engineers and the University of California have been used. It will be seen that, in general, the moduli of deformation of the field compacted samples agree more closely with those of samples prepared in the laboratory by kneading compaction than with those of samples prepared by impact or static compaction.

The relative values of the moduli obtained by the various laboratory compaction methods seem to be in general agreement with the results obtained in the tests previously described. The moduli for kneading compaction are consistently slightly higher than the moduli for impact

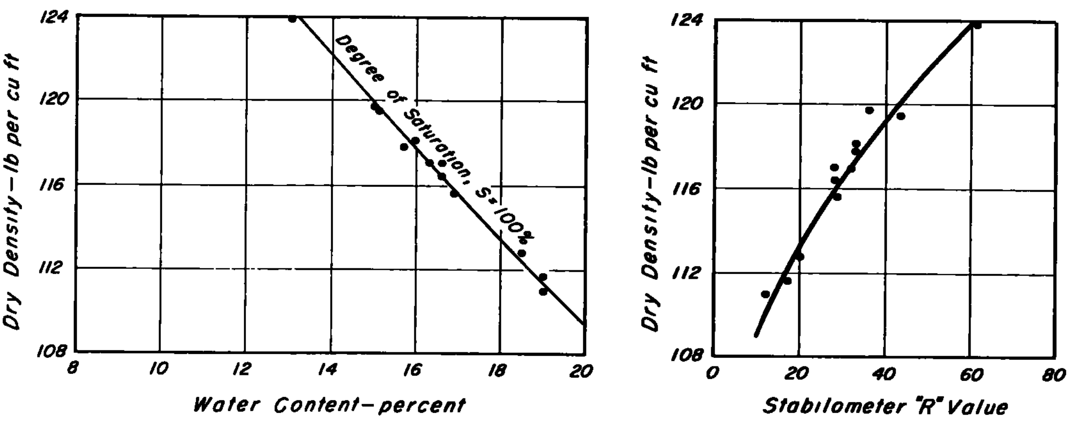

Figure 12d. Density versus water-content and density versus stability of samples prepared by kneading compaction and saturated by exudation.

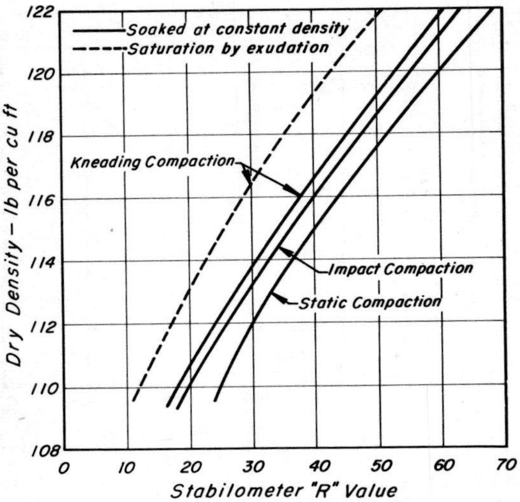

Figure 13. Density versus stability of samples soaked at constant density and samples saturated by exudation.

compaction; and although the moduli for static compaction are approximately equal to those for kneading compaction at the lower degrees of saturation, they are appreciably higher than those for kneading compaction at the higher degrees of saturation.

In general it may be concluded from these tests that static compaction appears to give the highest modulus values; impact compaction, the lowest values; and field and kneading compaction give intermediate values. While the results are by no means conclusive, they appear to substantiate previous indications that laboratory kneading compaction offers the greatest possibility for satisfactory duplication of field compaction effects.

EFFECT OF COMPACTION METHOD ON THE STABILITY OF SAMPLES SATURATED BY SOAKING

In the large majority of cases, pavement designs are based on the assumption that the compacted subgrade will become saturated at some stage in the life of the pavement which it supports. Thus, the design thickness is usually determined by the stability of a sample compacted in the laboratory and subsequently saturated by soaking at approximately constant density.

The effect of compaction method on the stability of samples treated in this manner was investigated by three series of tests on a sandy clay. In the first series, a number of specimens 4 inches in diameter and $4\frac{1}{2}$ inches high were prepared in metal molds by impact compaction at water contents both above and below the optimum. The density was determined and the upper $2\frac{1}{4}$ inches of each specimen was then trimmed off and used for water-content determination while the remainder of the specimen was maintained at constant density by confining it between two rigid porous plates and subjected to a water pressure of about 10 psi. at one of its ends. The water pressure was maintained until free water was seen to accumulate at the opposite end of the specimen, at which stage the specimen was considered to be saturated; the time required for this to occur was about 7 or 8 weeks. When all of the specimens in the series were saturated in this way, the water pressure was removed, the stabilities of the specimens were measured by Hveem Stabilometer tests, and the densities and water contents were determined. Similar series of tests were performed on samples prepared by kneading and static compaction, the compactive efforts being selected to give samples in the three series with approximately similar density ranges.

The results of these tests are shown in Figures 12a, 12b, and 12c. At the left of each figure is shown the density and water content of the sample as it was prepared

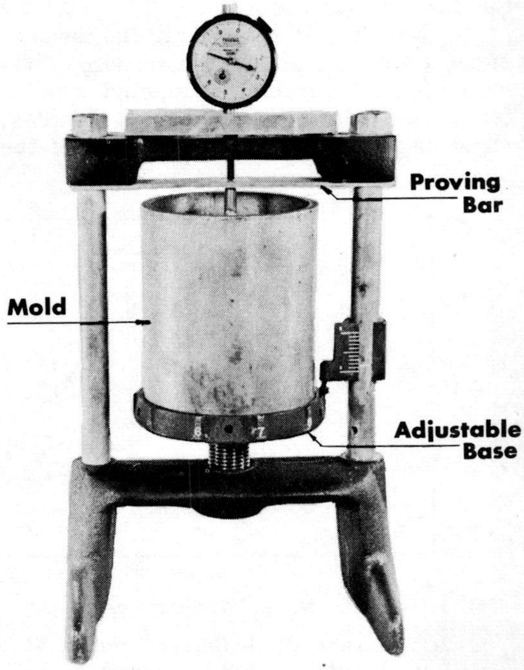

Figure 14. Expansion pressure device.

and after saturation, while on the right is shown the relationship between the resistance value and the dry density of the saturated specimens. It will be seen that only in isolated samples was a condition approaching complete saturation achieved, though the degree of saturation usually exceeded 95 percent. In general the average degree of saturation for samples prepared by impact compaction was slightly lower than that for samples prepared by static or kneading compaction.

The relationships of density versus stability after saturation for samples prepared by impact, static, and kneading compaction are compared in Figure 13. At equal densities, samples prepared by static compaction had the highest stabilities, and samples prepared by kneading compaction had the lowest stabilities. However, the stabilities for samples prepared by impact and kneading compaction did not differ greatly, and this difference might have been due to the slightly higher average degree of

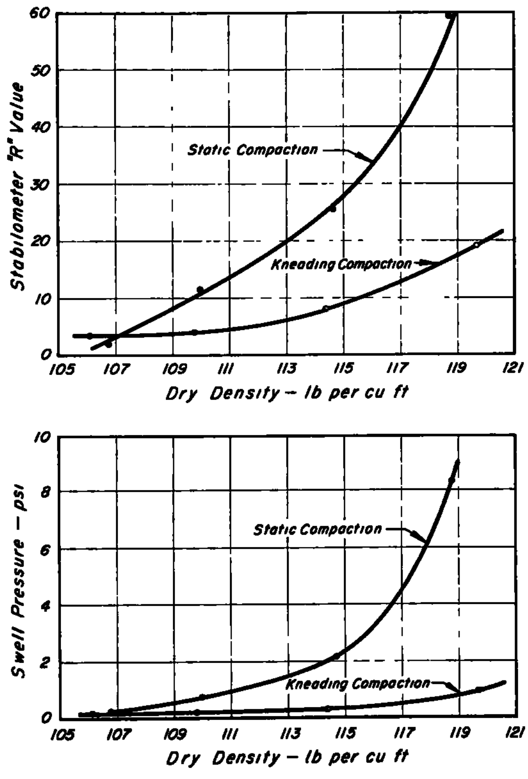

Figure 15. Density versus stability and density versus swell pressure of samples of silty clay prepared by static and kneading compaction and saturated by exudation.

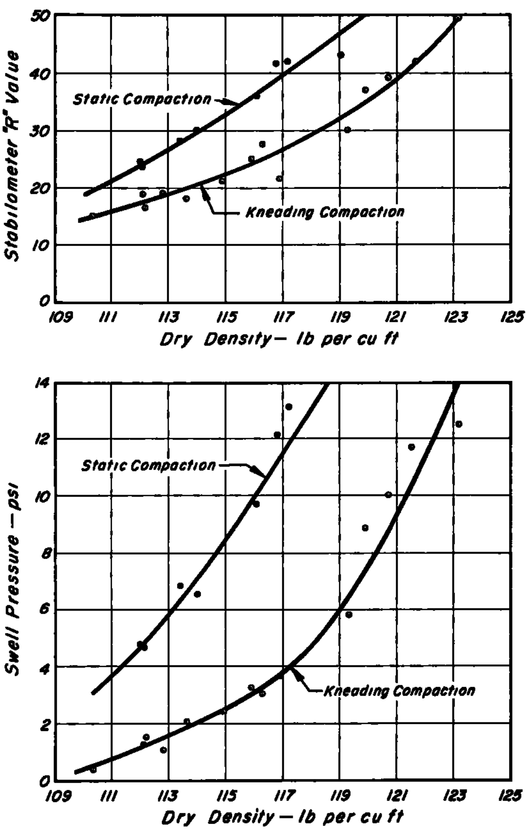

Figure 16. Density versus stability and density versus swell pressure of samples of sandy clay prepared by static and kneading compaction and saturated by exudation.

saturation for the samples prepared by kneading compaction. If the average degrees of saturation for samples prepared by impact and kneading compaction had been the same, the samples prepared by impact compaction might have had the lower stabilities. The stabilities, as measured by the resistance values of samples prepared by static compaction, were from 10 to 25 percent greater than those for samples prepared by kneading compaction. These results are in general agreement with the effects of compaction method on the stabilities of partially saturated soils.

In the California design procedure, samples are prepared by kneading compaction at water contents on the wet side of optimum and are then saturated by applying static pressure until moisture is exuded. It was considered of interest to compare the stabilities of samples prepared by this method with those obtained by the method of saturation previously described. The results of tests conducted in accordance with the

California design procedure are presented in Figure 12d, and the density-versus-stability relationship is shown in Figure 13. It will be seen that, for equal densities, the stabilities of these samples are considerably lower than those prepared by kneading compaction and saturated by soaking. While this difference may be due partly to the influence of the molding water contents on the stabilities of samples even after saturation, it may also be due to an increase in strength due to electrochemical action during the 8-week period of soaking the samples in metal molds. If this should be the case, the influence of this effect during long periods of soaking is clearly of considerable importance and indicates the need for special provisions for the saturation of samples after compaction.

EFFECT OF COMPACTION METHOD ON THE STABILITY AND SWELL PRESSURES OF SAMPLES SATURATED BY EXUDATION

Since the stabilities of samples saturated by long periods of soaking were considerably higher than those of samples saturated by exudation and were apparently affected by the test procedure, a further investigation was made to compare the stabilities of samples of two soils, a silty clay and a sandy clay, prepared by static and kneading compaction and then saturated by exudation of moisture under static load. Samples prepared in this way by kneading compaction are used for pavement design by the California Division of Highways, the samples being used to determine the stability of the soil and, also, the swell pressure exerted by the soil when it is confined in a mold and immersed in water; swell pressures are measured in the standard device shown in Figure 14.

The test procedure for each soil was essentially that used by California and may be outlined as follows:

Four or five samples at different water contents on the wet side of optimum were prepared in 4-inch-diameter metal molds by means of the Triaxial Institute Kneading Compactor, using 125 tamps and a tamping pressure of 350 psi. Each sample was then subjected to a static pressure applied at the rate of 600 lb. per min. until moisture was seen to be exuding from the base of the mold; at this stage the sample height was between $2^1/_4$ and $2^1/_2$ inches. After release of the pressure, the sample was allowed to stand for $^1/_2$ hour with the ends of the mold covered. A perforated metal disc with a vertical stem was then placed on top of the sample, and the mold was fitted in the swell pressure device. In this position the lower end of the sample rested on the adjustable base of the device, and the base was adjusted until the stem of the perforated plate on top of the sample was just tight against the proving bar. Water was then poured on top of the sample, and the subsequent deflection of the proving bar over a period of several days was measured by a dial gauge. The proving bars are relatively stiff, a pressure of 1 psi. exerted by the sample causing a deflection of only 0.003 inch; thus, only a slight expansion of the soil is permitted during the swell-pressure measurements. After the maximum swell pressure developed by each sample had been measured, the water was poured off the samples, the dimensions and weight of each sample were determined, and the stability was measured by a Hveem Stabilometer test. Finally the water contents of the samples were determined.

The entire process was repeated for samples at the same water contents, but compacted by static pressure, applied at the rate of 600 lb. per min., until moisture exuded from the base of the mold. The swell pressures and stabilities of these samples were determined as before.

The results of these tests on the silty clay are shown in Figure 15. It will be seen that for a density of about 107 lb. per cu. ft., which is the maximum density as determined by the standard AASHO compaction test for this soil, the resistance values and swell pressures of samples prepared by static and kneading compaction are about the same, but at higher densities samples prepared by static compaction have the higher resistance values and swell pressures. At a dry density of 117 lb. per cu. ft., which is the maximum density as determined by the modified AASHO compaction test for this soil, the resistance value of a sample prepared by static compaction is about 200 percent greater than that of a sample prepared by kneading compaction; and the swell pressure of a sample prepared by static compaction is about 800 percent greater than that of a sample prepared by kneading compaction.

The results of the tests on the sandy clay are shown in Figure 16. As for the silty clay, at equal densities, samples prepared by static compaction have higher resistance values and swell pressures than samples prepared by kneading compaction. For the sandy clay, the maximum density in the standard AASHO compaction test was 110 lb. per cu. ft., and the maximum density in the modified AASHO compaction test was 123 lb. per cu. ft. At a density of 110 lb. per cu. ft., the resistance value of a sample prepared by static compaction is about 30 percent greater than that of a sample prepared by kneading compaction, and the swell pressure is about 400 percent greater; at a density of 123 lb. per cu. ft., the resistance value of a sample prepared by static compaction is also about 30 percent greater than that of a sample prepared by kneading compaction, while the swell pressure is probably about 100 percent greater.

If, as the previous results would indicate, kneading compaction duplicates more closely the effects of field compaction equipment than static compaction, the erroneous values for desirable pavement thickness which would be obtained if designs were based on the results of tests on samples prepared by static compaction are immediately apparent.

PREPARATION OF SATURATED SAMPLES

In the tests described above it has been shown that the compaction of a soil to a saturated condition by the application of static pressure causes the stability and swell pressure of the soil to be higher than those for a similar sample prepared by kneading compaction and then saturated by the application of static pressure. If static pressure causes this difference in measured properties, the question is raised as to the extent to which it affects the properties of the samples prepared by kneading compaction. It would seem likely that the stabilities and swell pressures of samples prepared by kneading compaction and then saturated by the application of pressure until moisture is exuded would be somewhat higher than for samples of the same density, prepared by kneading compaction and then saturated without the application of static pressure.

However, the complete saturation of samples without the application of pressure is not an easy task. In one of the test series previously described an attempt was made to accomplish this by forcing water through the samples. Even after a number of weeks the samples were not completely saturated and the properties of the soil seemed to be affected in some way, possibly by electrochemical action resulting from the use of metal molds. This effect might have been eliminated by the use of plastic equipment, yet even if this were done, a waiting period of perhaps two months before a saturated sample could be obtained would be an undesirable situation for a testing laboratory processing a large number of samples.

Yet another method of saturation might be to soak a sample without necessarily preventing expansion. However, this process would also take considerable time and would probably lead to a nonuniform density and water content in the sample and to a higher degree of saturation at the ends than in the center section.

The preparation of completely saturated samples of clayey soils in the laboratory which will have the same properties as similar samples saturated in the field presents a number of problems, and it would seem that no simple method of accomplishing this has yet been developed. It may well be that the methods of saturation presently being used in various test procedures are entirely adequate for all practical purposes yet this cannot be ascertained until reliable test results for fully saturated soils can be obtained. It is regretted that no simple solution can be offered in this paper, but it is hoped that this brief discussion may stimulate further attention to this problem.

CONCLUSIONS

In methods of pavement design based on the results of tests performed on samples prepared in the laboratory, it is desirable that the test samples should have the same properties as those of the soil compacted in the field. The method of compacting a soil sample in the laboratory has a significant effect on the resulting properties of the soil. The main conclusions, with regard to the effect of compaction method on soil properties, resulting from the investigations described in this paper may be summarized as follows:

1. For the two soils investigated, a silty clay and a sandy clay, samples compacted to equal densities and water contents by kneading and impact methods do not show

any large difference in stability as measured by a Hveem Stabilometer test. At lower degrees of saturation, samples prepared by kneading compaction have higher stabilities and at higher degrees of saturation, samples prepared by impact compaction have higher stabilities.

2. At lower degrees of saturation, samples of the two soils investigated prepared by static compaction have somewhat higher stabilities, in stabilometer tests, than samples compacted by kneading and impact methods to the same densities and water contents; at higher degrees of saturation, samples prepared by static compaction have much-higher stabilities than similar samples prepared by impact and kneading compaction.

3. The moduli of deformation at 1 percent strain, as measured in triaxial-compression tests, for samples of a silty clay compacted in the field by sheepsfoot and rubber-tired rollers are in better agreement with those of samples at equal densities and water contents prepared in the laboratory by kneading compaction than with those of similar samples prepared by impact and static compaction.

4. Samples of the two soils investigated prepared by static compaction and saturated by soaking at constant density have considerably higher stabilities than samples of equal densities prepared by kneading and impact compaction and saturated by soaking at constant densitv.

5. For the two soils investigated, samples compacted by static pressure until moisture is exuded have much-higher stabilities and swell pressures, as measured by the test procedure of the California Division of Highways, than samples of the same density prepared by kneading compaction and then saturated by exudation.

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2. Seed, H. B. and Monismith, C. L. Some Relationships Between Density and Stability of Subgrade Soils. Paper prepared for presentation at the annual meeting of the Highway Research Board, January 11-15, 1954, Washington, D. C., Berkeley, California. : University of California, Institute of Transportation and Traffic Engineering, 1953. 12 pp. 13 Figs.

New Method for Measuring In-Place Density of Soils and Granular Materials

CARL E. MINOR, Materials and Research Engineer, and
HERBERT W. HUMPHRES, Senior Materials Engineer;
Washington State Highway Commission

Conventional methods of measuring the in-place density of soils and granular materials are time-consuming and oftentimes subject to considerable error. Presently used methods, such as measuring volumes by use of oil or calibrated sand, have one or more of the following shortcomings: (1) suitable only for application to a small range of hole sizes, (2) subject to considerable error when surface of ground is rough, (3) accuracy affected by vibration or temperature, (4) cannot readily check densities of successive lifts by merely extending depth of same hole, and (5) require lengthy calculations and correction factors for final answer.

In an attempt to overcome these deficiences, the Washington Densometer has been developed. The basic principle of the instrument is the same as that used in some other methods, namely that of inflating a rubber balloon with fluid until it fills the excavated hole and measuring the volume of the hole by measuring the amount of fluid so required. However, to the authors' knowledge, the apparatus is unique in the method of application of this principle.

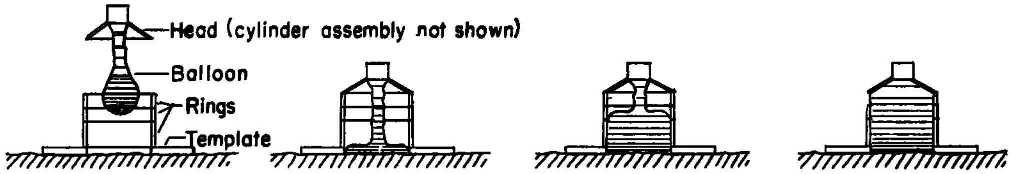
The device uses a closed system with a cylinder and piston to activate inflation and deflation of the balloon. This permits extremely rapid operation, plus the direct reading of volumes in cubic feet from the calibrated piston rod. By the inclusion of known-volume rings, rapid and accurate measurements of holes varying from 0.000 cu. ft. up to 0.500 cu. ft. can be made.

● THE need for a fast, accurate means of determining the field density of in-place granular base course and surfacing materials, as well as of subgrade soils, has become critical in recent years in the highway-construction field. Widespread application of moisture-density specifications to insure uniformity of roadbed construction requires that field methods be sufficiently rapid that control data can be made available to the engineer and the contractor during the compaction process and without causing delay to the progress of the job.

This is particularly critical in projects utilizing equipment-train methods of construction, such as soil-cement or cement-stabilized-base projects, where each phase of operation is integrated in time and order with other operations. The rapid progress of such projects does not allow sufficient time for determining field densities during the compaction phase using presently accepted methods. This has resulted in actually running "control" tests after a section of road is completed; the data serving primarily as a record to serve as a guide in future work. Often, the end result is a considerable waste of rolling time for compaction equipment, or in some cases, the need to disrupt the progress of the job because of the necessity of reprocessing a completed section.

Other phases of highway work have also been curtailed by the inadequacies of presently accepted standard field density test methods. In the field of research, this department has begun a long-range program of investigation of base-course and surfacing materials. A considerable number of pavement failures can be explained only in terms of failure of the granular surfacing materials and base courses. To determine what correction steps are required, it is necessary first to determine whether the deficiencies are in the materials themselves, or a result of the manner of placement or alteration of physical characteristic due to traffic. Extensive field studies will be required to find the correct answers, and one phase of these field investigations involves de-

A.



Balloon being lowered into rings for initial reading. Approximately 1 liter of fluid in balloon.

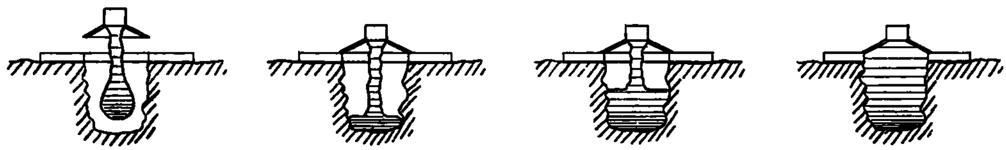
Condition of balloon with apparatus clamped in place prior to inflation.

During inflation. Note how balloon fills from bottom up.

Completely inflated. Record initial reading (IR). Deflate balloon and remove Densometer from rings.

B. Remove rings, dig hole, and leave out ring or combination of rings whose volume most nearly equals the estimated hole volume. Sum of volumes of rings left out = ring constant.

C.



Balloon being lowered into hole. Rings not on template.

Condition of balloon prior to inflation. Fluid in balloon causes bottom of hole to be filled without trapping air.

During inflation, filling from bottom allows balloon to fill voids displacing air upwardly and out vents.

Hole and head completely filled. Record final reading (FR).

D. Calculate volume: (FR + ring constant) − IR = volume of hole.

Note: The sketches above show a hole volume of approximately 0.2 cu. ft. and all rings were omitted for the final reading. For smaller holes, one or more of the rings would be used during the final reading. The purpose of the rings is to cause the balloon to be inflated to approximately the same degree for both the initial and final readings.

Figure 1. Sequence illustrating characteristics of densometer during operation.

termination of field densities both during the construction period and after various periods of traffic. The large volume of such data required dictates the need of field equipment that is both accurate and rapid. Presently accepted methods are not suitable for such studies for the reasons stated below, and this is one of the primary reasons the Washington Densometer was developed.

STUDY OF EXISTING METHODS

In 1952 this department reviewed the several existing methods generally accepted by various agencies for determining field densities, and found that each method possesses one or more of the following shortcomings: (1) required too much time for operation, exclusive of time required to dig hole; (2) suitable only for application to a small range of hole sizes close to the size of hole for which the apparatus is calibrated; (3) do not provide for initial surface reading, which introduces appreciable error when surface of ground is rough; (4) accuracy seriously affected by outside influences such as vibration, humidity, temperature; (5) will not permit checking densities of successive lifts by merely extending the depth of the same hole; (6) require frequent recalibration to insure accuracy; (7) require a level surface for successful operation; (8) not suitable for use in clean granular materials; (9) danger exists of trapping air in bottom of holes dug in fine-grained soils; and (10) require lengthy calculations and correction factors for final answer, which increases the possibilities for errors.

DISCUSSION OF WASHINGTON DENSOMETER

With these deficiencies in mind, development of the Washington Densometer was started. Successive improvements in design resulted in a device that has none of the shortcomings listed above. The schematic drawing shown in Figure A illustrates the basic features of the apparatus. The principle utilized is the same as that used in some other methods; namely, that of inflating a rubber balloon with fluid until it fills the excavated hole and measuring the amount of fluid so required to determine the volume of the hole. However, to the writers' knowledge, the apparatus is unique in the method of application of this principle. The use of a closed system with a cylinder and piston activating the inflation and deflation of the balloon permits extremely rapid operation and the direct reading of volumes in cubic feet from the calibrated piston rod. The inclusion of known-volume rings permits the use of a large-size balloon for both small and large holes without loss of accuracy. The purpose and use of these rings are illustrated in Figure 1.

In summary, the following features and advantages are found in the densometer method of measuring hole volumes:

1. Operation time is sufficiently short that "wet densities" can be determined in from 10 to 20 minutes depending on the time required to dig the hole. Of the total time consumed, only about 3 minutes are required for setting up the densometer and making the necessary readings.

2. An initial reading that will account for variations in roughness of the original ground surface at the hole site is taken, which increases the accuracy of the determination.

3. Volumes of holes from 0.000 cu. ft. to 0.250 cu. ft. can be measured with equal accuracy. (Actually, this volume can be doubled by recharging the cylinder during the operation). The large capacity of the device makes it applicable for determining densities in coarse, granular materials as well as in fine-grained soils.

4. The balloon membrane is very thin, and the size of the balloon is such that it fills the excavated hole without being stretched appreciably. These features,

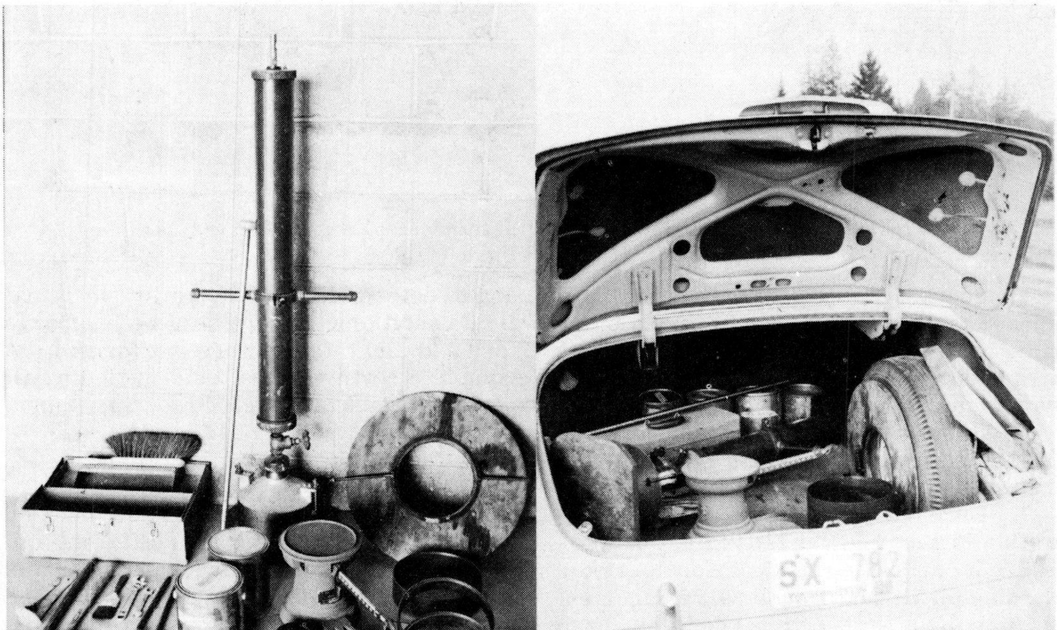


Figure 2. Complete field kit for Washington Densometer; (left) including tools and equipment for field maintenance and operation; (right) loaded for transport in trunk of passenger car.

coupled with the fact that the balloon is lowered into the hole in such a manner that the hole is filled from the bottom up, insure that air is not trapped and that practically all pits and voids in the hole walls are filled.

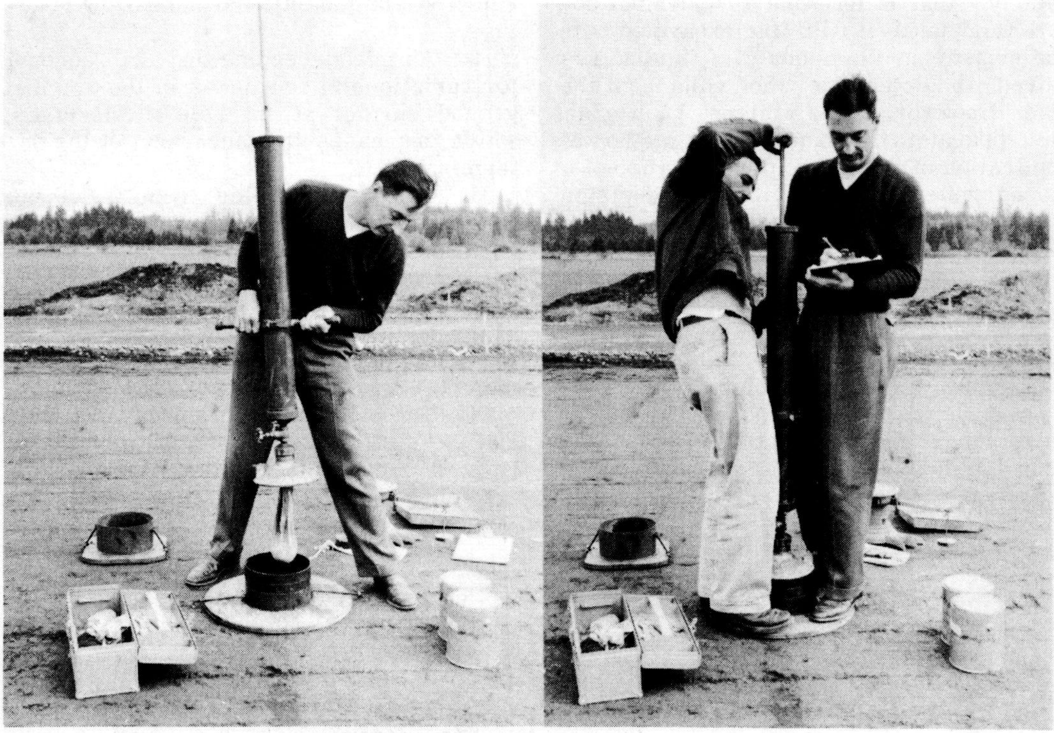


Figure 3. Lowering balloon into rings and template (left) and taking initial readings (right).

5. Successive determinations on any number of thin layers can be made in the same hole. Holes up to 18 inches in depth have been tested; however, the anticipated maximum depth of hole is about 22 inches.

6. Results obtained are of equal or greater accuracy than those obtainable by presently accepted standard field methods.

7. Once calibrated, no further calibration is necessary. Little field maintenance is required, except for periodic bleeding off air and occasional replacement of the balloon. These procedures have been simplified to a point where very little operational time is lost.

8. Readings are made directly in cubic feet, which greatly simplified calculations.

9. The apparatus is of such size that it can be carried in the trunk of a passenger car, and is sufficiently light that one man can carry it with no difficulty (see Fig. 2).

RESULTS OF EXPERIMENTS

To verify the accuracy of the densometer and to determine its adaptability to actual field conditions, a series of both laboratory and field tests were performed. A complete review of the tests including all pertinent data is appended to this paper.

LABORATORY EXPERIMENTS

1. A series of readings was made on the same hole site by two different operators. It was found that the apparatus exhibits a definite stopping point when the piston is depressed; that an operator will repeat his determinations accurate to less than 0.0002 cu. ft.; and that two different operators can repeat each other's determinations with equal accuracy.

2. A container whose volume was accurately determined by precise water-filling methods was checked with the densom-

eter. The volume determined agreed exactly with the true volume on two trials, and varied 0.0001 cu. ft. on one trial. Different operators were used for the third trial. This accuracy greatly exceeds that required for field density tests.

comparative results were obtained. The densometer consistently gave slightly larger hole volumes for the same hole, and the difference increased as the roughness of the hole surface increased. This indicates that the densometer more-completely fills the voids than does the sand.

Figure 4. Balloon, flexible suction tube and tapered sleeve, carrying base, head, lock ring, wrench. Upper right shows balloon slipped over suction tube and sleeve and being placed in head. Lower left shows head clamped to carrying base and lock ring being tightened to seal balloon in place. Lower right shows cylinder in position on head and being tightened with special spanner wrenches.

3. A series of parallel tests was run with the sand-volume method and the Densometer. It was found that if extreme care was used in calibrating and operating the sand-volume apparatus, and if an initial ground-surface reading was made with the sand-volume apparatus, close

FIELD EXPERIMENTS

1. The densometer was used for field control of compaction of cement-treated and granular base materials on five dif-

P SH No. 9 Cont. No. 4490 Section Bogachiel to Hoh River

Inspector Humphres - Gadbois Date Aug. 12, 1953

Test Hole No.	7						
Station	723+00						
Reference to ℄	5' Left						
Reference to Grade	C.T.B.						
DENSITY DETERMINATION							
Can No.	1						
Densometer Final Reading "A"	0.1712						
Ring Constant "C"	0.1500						
Sum of Final Reading plus "C"	0.3212						
Densometer Initial Reading "B"	0.1923						
Vol. of Hole-Cu.Ft.(A+C-B)	0.1289						
Wt. Wet Soil from Hole + Can	19.87 lb						
Wt. of Can	1.49						
Wt. Wet Soil - lbs.	18.38						
Wet Density - lbs./cu.ft.	142.6						
Dry Density - lbs./cu.ft.	134.3						
Max.Density - lbs./cu.ft.	136.1						
% of Max.Density-lbs./cu.ft.	98.7						
MOISTURE DETERMINATION							
Wt. of Damp Soil + Tare	4.38						
Wt. of Dry Soil + Tate	4.25						
Wt. of Moisture	0.13						
Wt. of Tare	2.15						
Wt. of Dry Soil	2.10						
% Moisture	6.2						

$$\text{Wet Density} = \frac{\text{Wt. of Wet Soil}}{\text{Vol. of Hole}}$$

$$\text{Dry Density} = \frac{\text{Wet Density}}{1 + \frac{\%\ \text{Moisture}}{100}}$$

$$\%\ \text{Max.Density} = \frac{\text{Dry Density} \times 100}{\text{Max. Density}}$$

$$\%\ \text{Moisture} = \frac{\text{Wt. of Moisture} \times 100}{\text{Wt. of Dry Soil}}$$

1 Gram = 0.0022 lbs.

1 lb. = 453.6 grams

Remarks: #7 Hole 6" deep in cement-treated Base Course. Rolled for 1-hr w/ pneumatic roller and two 10-Ton tandem steel-wheeled rollers. Appearance - excellent

Figure 5. Suggested form with sample data for field density test report.

ferent projects. On one project parallel determinations were made with the sand-volume method. On this project results of the two methods agreed closely (±0.0004 cu. ft. for holes approx. 0.10 cu. ft. in volume) except occasionally when construction equipment working nearby vibrated the ground while running the sand-volume determination. Differences then increased to a maximum of 0.0015 cu. ft. Intermittent checks made on the other projects gave similar results. Speed trials using the two methods showed that it was possible to complete two determinations of wet density using the densometer while completing one with the sand. The trials included the time required to clean the hole and refill it with fresh cement-aggregate mix and the time required to calculate results.

An alcohol-burning method of drying the samples was correlated with oven-drying, and was used on the grade to determine moisture content in conjunction with the densometer. With two men and using this procedure, it required approximately 10 to 15 minutes to determine the wet density, and an additional 5 minutes to determine the dry density. At no time was it necessary to delay the operations of the contractors for density information. It was consistently possible to check at least one density, and sometimes two or three densities in each 400-foot to 800-foot section of roadway before the rollers were needed on the next section. It was found possible to complete a wet-density determination for each pass of a roller when the sections were over 600 feet long without interfering with the roller operation.

On one of the above projects, it was necessary for one man to work alone temporarily. Using the densometer and the alcohol-burning method of drying, he was able to make six complete dry density determinations in 3 hours. These tests were made in an 8-inch lift of 1 inch of clean crushed gravel. On several occasions on this project, tests were made on successive 4-inch layers in the same hole with excellent results.

2. The densometer has been used to check the densities of crushed stone surfacing, top course. Three densities were taken at each of several sites at different offsets from centerline. The alcohol-burning method was used for determining moisture content. Moves at about 3,000 feet were made between test sites. It was possible to make up to 30 complete determinations per 8-hour day under these conditions. Results were very consistent and logical. Occasional comparisons with the sand-volume method gave good results and verified the accuracy of the densometer.

SUMMARY

In actual field use, the Washington Densometer has proven itself to be considerably faster, equally or more accurate, and extremely more versatile than other existing accepted types of equipment designed for measuring in-place hole volumes in soils.

The accuracy, versatility, and speed attainable with this equipment have greatly increased the practical degree of actual control possible on controlled-compaction projects. The range of material types for which control can be considered practical has been broadened considerably, and the effectiveness of various types of rollers on thin or thick lifts of soil can now be readily investigated.

Maintenance requirements of the equipment during use has proven to be practically negligible. The balloon mortality rate is lower than anticipated, with replacements being required only after the equipment has been stored unused for considerable time.

Reception by field inspectors, resident engineers, and contractors has been favorable.

Appendix A

DESIGN AND OPERATION PROCEDURE

The schematic sketches and section-plan drawings (Figs. A-D) show the physical arrangement and dimensions of the apparatus. It is designed to handle hole volumes up to approximately 1/4 cu. ft. without recharging the cylinder, or up to approximately 1/2 cu. ft. or more with recharging. Tests to date have been confined to 1/2 cu. ft. and less, so no statement of satisfactory performance at volumes above that can be made at this time.

The piston is designed to move freely in the tube, and yet give a sufficiently tight seal to allow the fluid to be sucked upward into the tube without loss of seal. The leather seals were selected after trying several materials and have worked very well. They tend to freeze temporarily on setting unused for long periods, but can be freed by light tapping on the handle, or by turning the rod with a wrench, and will then work freely and smoothly.

The balloon used was selected primarily for its size, which allows digging holes of about 9 inches in diameter, a desirable size for granular materials. It has a thin wall, and a high stretch ratio, which allows it to conform to small irregularities in the hole surface. Being a standard weather balloon, it is commercially available at low cost.

Soluble oil, the type used by machinists to make cooling water, is added to the water used in the apparatus to avoid rust and corrosion, and to lubricate the cylinder walls. Other fluids, such as hydraulic brake fluid, could be used, but the water-oil solution is inexpensive and satisfactory.

The detachable handle is calibrated in divisions of 0.001 cu. ft., and readings

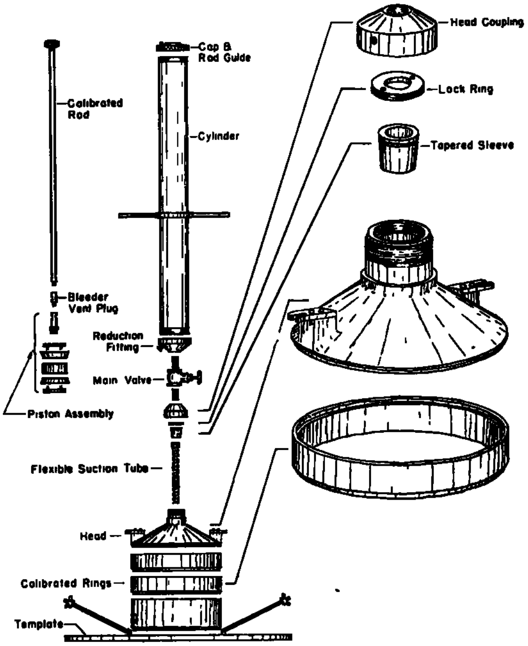


Figure A. Schematic assembly sketch.

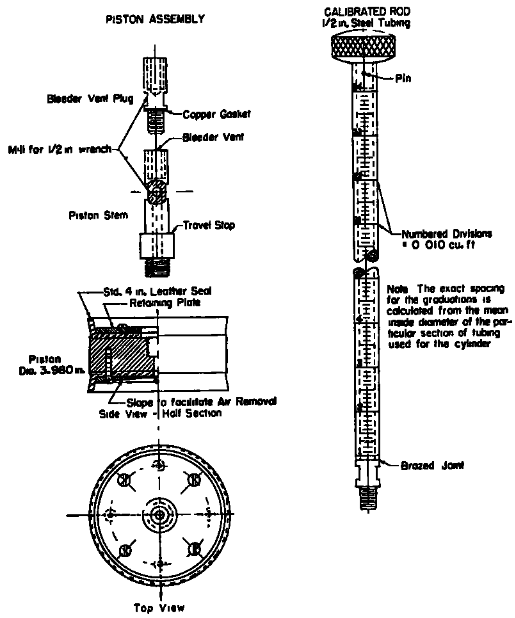


Figure C.

accurate to 0.0002 cu. ft. are made by interpolation between divisions. With the handle removed, the Densometer will fit easily into the back seat or trunk of a car. The carrying base is clamped to the head to protect the balloon during transportation, and also serves as a stand for the apparatus between readings during operation (see Fig. 2).

The calibrated rings are machined as closely as possible to volumes of 0.0500 cu. ft., and 0.1000 cu. ft., the actual volumes, both singly and in various combinations, being established by careful check with water or other precise methods. Once calibrated, these volumes are established as constants for the apparatus and so marked on the respective rings. Change of the ring volumes due to normal tempera-

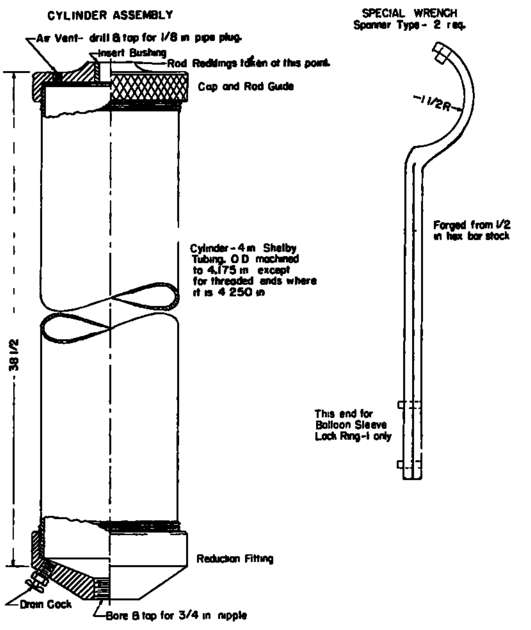


Figure B.

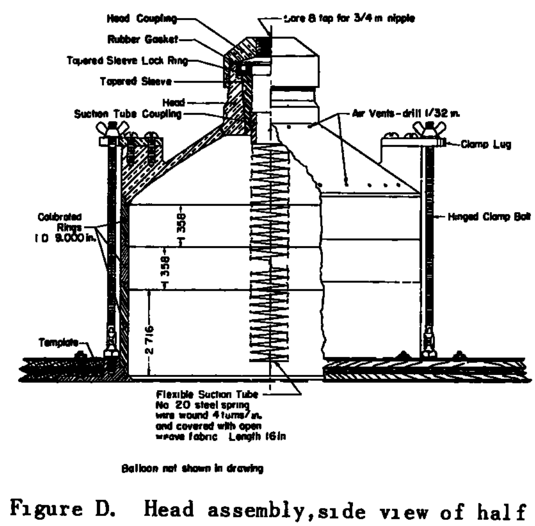


Figure D. Head assembly, side view of half section.

ture fluctuations are not of sufficient magnitude to introduce appreciable error in final results. However, if the apparatus is to be used in areas where extreme temperature changes are common, it would be good practice to establish a volume-temperature calibration curve for the device. It has been established that temperature fluctuations of less than 30 F. have no noticeable effect on final results. Further investigative work of temperature fluctuations is contemplated.

OPERATING PROCEDURE

A. Assembly and filling with fluid. (Necessary occasionally when balloon is replaced).

1. Remove head from cylinder at joint below main valve and remove tapered balloon sleeve and flexible suction tube (Fig. 4).

2. Slip neck of balloon over tube and tapered balloon sleeve until flare of balloon neck is flush with bottom of tapered sleeve.

3. Trim off neck of balloon at or slightly below groove near top of balloon sleeve. Lower deflated balloon through opening in head until the tapered sleeve is firmly seated.

4. With head clamped to carrying base, place tapered sleeve lock ring in position and tighten, forcing tapered sleeve into final seated position.

5. Place head gasket in head coupling and connect the upper cylinder to the head.

6. With densometer clamped to carrying base, and piston removed, open main valve and fill with fluid until about 6 inches of fluid raises in cylinder. Rock gently to facilitate passage of air bubbles from balloon and head upward into cylinder.

7. Insert piston, bleeder vent open, and push down until fluid raises in vent hole. Close piston bleeder vent with calibrated rod and raise piston, sucking fluid from balloon into cylinder. Leaving approximately 1,500 cc. of fluid in balloon, close main valve.

8. Remove piston. (If piston is too low in cylinder to allow removal by opening piston vent, open drain valve at bottom of cylinder while pulling up piston). Adjust fluid level to within about 3 inches of top and replace piston. Cover top of piston with enough fluid to cover the leather seals, install bleeder vent plug and cylinder cap.

9. Inflate balloon in carrying case several times by opening main valve and operating the piston to insure removal of all air from balloon. After last run, close main valve and exert a strong upward pull on piston to de-air the fluid in the cylinder.

10. Remove bleeder vent plug and press piston down until fluid raises to top of vent. Replace plug. The apparatus is now de-aired and ready for use. (This step should be repeated whenever Densometer has not been in use for some time).

Although the above assembly operation appears quite involved, it is quite simple in practice, and can be completed in approximately 10 minutes.

B. Field Density Determination:

1. After selecting site, smooth ground sufficiently so that template sets solidly. Do not attempt to level surface that will lie within center area of template. Place rings on template.

2. Place Densometer on template and rings, and clamp in position (Fig. 3). With two men standing on template for rigidity, open main valve and apply gentle pressure to calibrated rod. Sufficient pressure should be used to cause piston to lower at a constant, medium rate. When the balloon fills the rings and head, a definite shock will be felt. Maintaining slight pressure on handle, note reading on rod at top of cylinder cap. Repeat three times by lifting and depressing piston about 10 inches to ascertain that no air is trapped and apparatus is functioning properly. This will be shown by the consistency of the readings. (No trouble has been experienced from this source to date). Record reading as "Initial Reading". Note: One man can operate Densometer if heavy bag of dirt is used to balance load on template.

3. Evacuate fluid from balloon by lifting piston, close main valve, remove Densometer, and set on carrying base.

4. Remove calibrated rings from template, and, being careful not to move template, dig hole as for other methods. Save all material in a sealed can to avoid loss of moisture. The excavated hole should be as smooth as possible, avoiding small radius holes or extreme undercutting of the template.

5. Upon completion of hole, leave out whatever number of calibrated rings are estimated to most nearly equal the

volume of the hole, and clamp Densometer to template. (Care should be taken to lower balloon into hole in such a position that it will not be twisted when the Densometer is seated.) Repeat Step 2, and record reading as "Final Reading". Repeat Step 3.

6. Add volumes of rings left out in Step 5, and record as "Ring Constant". Determine volume of hole by following formula:

(Final reading + Ring Constant) - Initial Reading = Vol. of hole in cu. ft.

7. Weigh wet soil taken from hole and calculate wet density:

$$\frac{\text{Wt. wet soil (lb.)}}{\text{Vol. of hole (cu. ft.)}} = \text{Wet density (lb./cu. ft.)}$$

8. Determine moisture content by accepted means and calculate dry density.

C. Determination of volume of holes larger than 0.230 cu. ft.

For such holes, it is necessary to recharge the piston to furnish sufficient fluid to fill the hole. The following procedure may be used:

1. See B-1 above.
2. See B-2 above.
3. See B-3 above.
4. See B-4 above.
5. Upon completion of hole, leave out all of the rings and clamp Densometer to template. Open main valve and allow fluid in cylinder to pass into balloon until piston is just short of extreme lower position. Close main valve and record reading as F.R. No. 1.
6. Open drain valve to allow air into cylinder, and raise piston to extreme top position. Close drain valve, remove handle, bleeder vent plug, and cylinder cap, and remove piston from cylinder.
7. Fill cylinder with additional fluid, replace piston, de-air and reassemble cap, plug and handle. Record reading as I.R. No. 2.
8. Open main valve and complete filling of holes as in step B-2 above. Record final reading as F.R. No. 2.
9. Fluid is evacuated as in step B-3 above, except that excess fluid is eliminated through the drain valve into a container.
10. Calculate hole volume as follows: (See Fig. 5).

(F.R. No. 1 - I.R. No. 1) + (F.R. No. 2 - I.R. No. 2) + 0.2000 = Vol. in cu. ft.

Note: Minor modification of the apparatus to include a filling tube connected to the bottom of the cylinder and rising along the side will permit recharging the cylinder without removing the piston. Details of this modification are now being worked out and will be published as an addendum to this report when completely tested.

Appendix B

RESULTS OF EXPERIMENTS

1. Check for consistency of readings.

Initial readings taken with all three rings in place. Final reading taken after removal of specified ring. All readings taken at same position on smooth, flat concrete slab.

Note: Volumes indicated are slightly smaller than true volume of ring removed because balloon is not inflated to same degree for both readings.

Series No. 1 (Ring No. 1 removed for final reading)

I. R.	0.1862	0.1860	0.1862	0.1861
F. R.	0.1366	0.1364	0.1365	0.1365
Vol.	0.0496	0.0496	0.0497	0.0496 cu. ft.

Bled air before start of second series.

Series No. 2 (Ring No. 2 removed for final reading)

I. R.	0.1970	0.1969	0.1969	0.1970
F. R.	0.1475	0.1475	0.1474	0.1474
Vol.	0.0495	0.0494	0.0495	0.0496 cu. ft.

Series No. 3 (After setting in sun 1 hr., temp. 86 F.; Ring No. 2 removed for final reading)

I. R.	0.1954	0.1953	0.1953	0.1953
F. R.	0.1458	0.1458	0.1458	0.1458
Vol.	0.0496	0.0495	0.0495	0.0495 cu. ft.

Bled air before start of series 4.

Series No. 4 (Ring No. 3 removed for final reading)

I. R.	0. 2233	0. 2233	0. 2233	0. 2233
F. R.	0. 1238	0. 1238	0. 1238	0. 1238
Vol.	0. 0995	0. 0995	0. 0995	0. 0995 cu. ft.

Series No. 5 (Rings No. 1 and 2 removed for final reading)

I. R.	0. 2244	0. 2244	0. 2244	0. 2244
F. R.	0. 1254	0. 1254	0. 1254	0. 1254
Vol.	0. 0990	0. 0990	0. 0990	0. 0990 cu. ft.

CONCLUSION: Apparatus exhibits a very definite stopping point which will repeat itself consistently. Variations of 0. 0002 cu. ft. are possible in making individual readings due to human variations in interpolating the scale. When initially filled with water, air should be bled frequently, as a small amount of air will separate each time fluid is pulled into piston. This effect becomes practically negligible after several runs. Normal temperature changes do not appreciably affect the accuracy of the apparatus.

2. Calibration of rings: Rings were calibrated by a careful, exact water-volume method. One ring was sealed to a rubber base, leveled, and filled with water, using a glass plate to establish when full. The other rings were then added, the joints being sealed with tape, and their volumes determined successively, the procedure was then reversed to establish the volume of the bottom ring. Three checks were made, and temperature corrections were included. The following volumes were determined:

Ring No. 1 = 0. 0498 cu. ft.
Ring No. 2 = 0. 0500 cu. ft.
Ring No. 3 = 0. 0998 cu. ft.

By volume of rings in pairs, it was established that each joint increased the volume 0. 0002 cu. ft., and:

Rings No. 1 and 2 = 0. 1000 cu. ft.
Rings No. 2 and 3 = 0. 1500 cu. ft.
Rings No. 1 and 3 = 0. 1498 cu. ft.
Rings No. 1, 2, and 3 = 0. 2000 cu. ft.

For practical purposes, the constants used were established as 0. 0500 cu. ft. for each small ring, and 0. 1000 cu. ft. for the large ring.

3. Check of Known Volume: A cylindrical metal container with a rounded bottom joint was used, and its volume determined by the water method. Initial readings were taken on a smooth, flat concrete surface with the densometer, and then the template was set on the container, rings No. 2 and 3 removed, and the final reading made after filling the container. Cylinder was bled between each trial to give different set of readings. Established volume of container = 0. 1675 cu. ft.

	Trial 1	Trial 2	Trial 3
F. R.	0. 2127	0. 2175	0. 2230
Ring Constant	0. 1500	0. 1500	0. 1500
Sum	0. 3627	0. 3675	0. 3730
I. R.	0. 1952	0. 2001	. 2055
Vol.	0. 1675	0. 1674	0. 1675

CONCLUSION: The device will accurately determine the volume of a hole. Operators were changed for each of the above trials, and part of the fluid was bled out of the cylinder to avoid repetition of readings. The trials show that excellent results are obtainable independent of the operators.

4. Comparison to sand cone method (California large sand-volume apparatus) in actual field conditions: In these tests, it was found imperative to use extreme care in making the sand cone determinations to get consistent results. The cone was calibrated by the water method, and the sand was calibrated frequently during the tests. To eliminate error due to uneven ground surface, initial readings were made, using a template and a thin rubber membrane (dental dam) so the sand could be recovered and weights were determined accurately to 0. 01 lb. Parallel determinations were made on the same hole with the sand cone and the Densometer. The following results are typical of this series of tests:

Vol. (Sand Cone)			
1	2	3	4
0. 0906	0. 1120	0. 0878	0. 1057 cu. ft.
Vol. (Densometer)			
1	2	3	4
0. 0907	0. 1124	0. 0880	0. 1058 cu. ft.

Note: The Densometer consistently gave slightly large hole volumes for the same hole, and the difference increased as the roughness of the hole surface increases. This indicates that the Densometer fills the small pits and irregularities more completely than does the sand.

Occasionally it was not possible to avoid having construction equipment working nearby. On these occasions, the volume determined by the sand cone was consistently larger than that determined by the Densometer. The degree of error caused by vibration of the sand is indicated by the following typical results:

	1	2	3	
Vol. (Sand Cone)	0.1192	0.1289	0.1692	cu. ft.
Vol. (Densometer)	0.1180	0.1277	0.1674	cu. ft.
Error	0.0012	0.0012	0.0018	cu. ft.

5. Time trials: Actual operation of the Densometer including set-up time, but excluding hole digging time is about 3 minutes. Under actual field conditions, where it was necessary to clean up the hole and re-fill with compacted soil, approximately two complete determinations could be made with the Densometer while making one with sand-volume apparatus.

Including average hole digging time, wet density determinations require from 10 minutes to 15 minutes under actual field conditions. The density of 2-inch to 4-inch layers of crushed stone surfacing was determined on several projects during construction. The alcohol-burning method was used to determine the moisture content, and it was possible for a two-man crew to make as many as 30 complete dry density determinations in an 8-hour day. Three tests were made at each location, and moves of about 1,000 feet were made between locations.

On one project, density determinations were made in an eight-inch lift of clean crushed gravel base course material. One man working alone was able to complete six dry density determinations in three hours.

6. Measuring Large Holes: Two holes were dug 17 inches and 18 inches in depth respectively. The volume of each was checked with the Densometer and by the sand-volume method using the large California sand-volume apparatus. Careful calibration of the sand apparatus was made for each determination. Initial readings were taken with both types of equipment.

	Hole #1 (17" deep)	Hole #2 (18" deep)
Densometer	0.4853 cu. ft.	0.5081 cu. ft.
Sand Cone	0.4830 cu. ft.	0.5060 cu. ft.
Difference	0.0023 cu. ft.	0.0021 cu. ft.

Results show good agreement and difference would affect a normal density less than 1 lb./cu. ft. The slightly larger volume shown by the Densometer indicates more complete filling of the voids.

Discussion

ALFRED W. MANER, Highway Research Engineer, Virginia Council of Highway Investigation and Research — The authors are to be congratulated for developing a piece of equipment that permits rapid and accurate determination of field densities. With the Washington Densometer they seem to have overcome to a great extent the deficiencies of the more-common methods of measuring field densities.

Of prime importance is the reduction of time for operation. Undoubtedly we shall someday have equipment that will measure moisture and density in a very few minutes without disturbing the soil or other material, but to have a device now that is fast enough to permit control during the rolling operation is a big step forward.

It is particularly interesting to learn that the volume of relatively large holes can be determined with the Washington Densometer. During the past year we had occasion, in Virginia, to determine the relative densities of two sections of waterbound macadam compacted by two different methods. Taking a cue from North Carolina we used a heavy, 15-inch-diameter ring as a guide in digging the hole, a 100-lb. bag of Ottawa sand for measuring the volume, large buckets and platform scales for weighing, in addition to other necessary equipment. We would have welcomed a device like the densometer at that time.

In using the densometer for determining the density of different layers in the same hole, it seems that extreme care must be exercised in order not to change the volume of the hole in the upper layers when digging in the lower layers. It would be easy to change the volume of the hole after the determination has been made and thus give erroneous results in succeeding layers.

Effect of Repeated Load Application on Soil Compaction Efficiency

GEORGE F. SOWERS, Professor of Civil Engineering, and
C. M. KENNEDY III, Graduate Student;
Georgia Institute of Technology

Soil compaction is essentially a process of consolidation. Our knowledge of that process leads to the conclusion that the greatest amount of soil consolidation is produced by the fewest cycles of application of load. This has been demonstrated by recent research at the Georgia Institute of Technology. Different methods of compaction such as tamping, hammering, and squeezing were employed, each exerting the same amount of work. The fewer the applications of pressure neccessary to exert the same amount of work, the greater the density obtained.

●IN the past, soil engineers have emphasized the need for controlling and evaluating compaction and have largely left the problem of obtaining it to the contractors and equipment manufacturers. The few studies which have been made, such as those by the U. S. Waterways Experiment Station (1) and the Road Research Laboratory in England (2) have done much to point out how better compaction can be obtained easily. Much research remains to be done, however, on the basic factors which control the effectiveness of compaction. It is the authors' purpose to discuss just one aspect of the problem, the effect of repeated load applications on compaction efficiency, and to point out how this affects both laboratory and field compaction results.

COMPACTIVE EFFORT AND COMPACTION

The amount of work exerted in compacting a soil is the compactive effort. It may be described by the number of blows of a certain weight hammer falling a fixed distance, the number of applications of a certain pressure to the soil surface, or by the number of passes of a roller of known weight and pressure over a specified lift of fill. Technically it is most accurately expressed by the work in foot-pounds or inch-pounds applied to each cubic foot of soil.

A number of field and laboratory studies of compaction utilizing different amounts of work applied have come to the same conclusion: that soil density increases as the compactive effort increases for any given soil condition. The relationship is not linear, however, for the rate of increase in density decreases with increasing work. For example, research by the Waterways Experiment Station (1) indicates that a linear relationship exists between dry density and the logarithm of the number of hammer blows in a laboratory test or the logarithm of the number of passes of a sheepsfoot roller.

Little has been said about the effect of the way in which the compactive effort is applied to the soil. In fact, some investigators have implied that soil density will always be about the same for a given effort in foot-pounds per cubic foot regardless of the manner in which the effort is exerted.

Others have pointed out that considerable differences in compaction effectiveness do exist. For example, moisture-density curves developed by the Standard AASHO procedure (Standard Proctor Test) do not necessarily have the same shape as the moisture density curves developed by actual rolling in the field. The difference has been attributed to the fact that the standard laboratory test involves tamping or dynamic compaction while the compaction of a sheepsfoot roller is produced by an increasing static pressure. Some laboratory procedures have been developed to simulate the action of the sheepsfoot.

Research in the Soil Mechanics Laboratory at Georgia Institute of Technology (3) has shown considerable difference between the densities produced by different compaction methods even though all utilized the same total compactive effort. For example, identical soil samples were compacted using first, the Standard AASHO method, and sec-

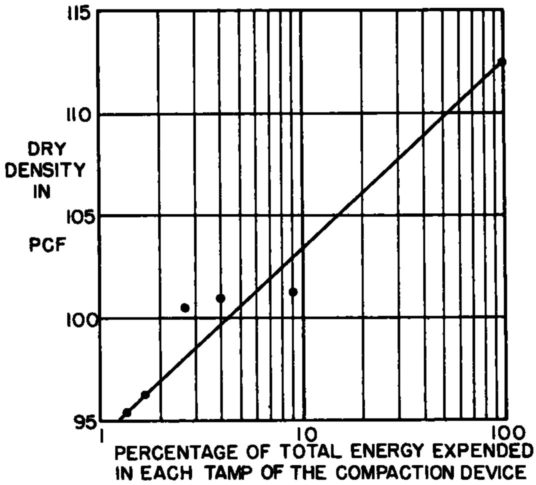


Figure 1. Relationship between the compacted density of the soil and the percentage of total energy exerted by each tamp of the compaction device (typical example).

ond, single applications of static pressure to each of three layers of soil in a standard compaction mold. The second method was adjusted by trial and error to utilize the same amount of total effort as the first. The density obtained by the first procedure was 107 pcf. and by the second method was 119 pcf. This indicates that the second method is more effective in utilizing the work done in producing compaction.

EXPERIMENTAL STUDY OF FACTORS AFFECTING COMPACTION EFFECTIVENESS

A program of study was undertaken at Georgia Tech to determine the factors which affect compaction effectiveness (3). The work was carried out with a single soil type (low plasticity clay, A-6), a limited number of moisture contents, and a single compactive effort of 34,000 foot-pounds per cubic foot. The latter was selected as it is between the standard and the modified AASHO efforts and is representative of modern compaction requirements.

A number of different devices were used. These included hammers weighing 5.5, 10, and 25 lb. with heights of fall ranging from 3 to 18 inches, a low velocity punching tamper to simulate the action of a sheepsfoot roller foot, and a piston for applying slow, static pressure to the soil. The results of 64 tests indicate that for moisture contents above the standard Proctor optimum all methods produced substantially the same densities while at moisture contents equal to or below the optimum, the densities were quite different.

A number of factors were considered which might be the cause of the difference. These included: (1) velocity of hammer at point of impact; (2) momentum of the hammer; (3) hammer weight; (4) ratio of the diameter of the compacting device to the thickness of the soil layer; and (5) percentage of the total energy exerted during each tamp or application of pressure.

The results of these tests indicate that the velocity of the hammer or tamper as it strikes the soil has no discernible influence on the effectiveness of compaction. Neither do the momentum or the weight of the hammer. These results contradict the conclusion reached by some investigators that the difference between the standard laboratory tests and field results lies in the fact that the laboratory tests are essentially dynamic (with high velocity of impact) while the field work is essentially static (with low velocity of impact).

The ratio of the hammer or tamper diameter to the soil layer thickness was found to be an important factor. Research is con-

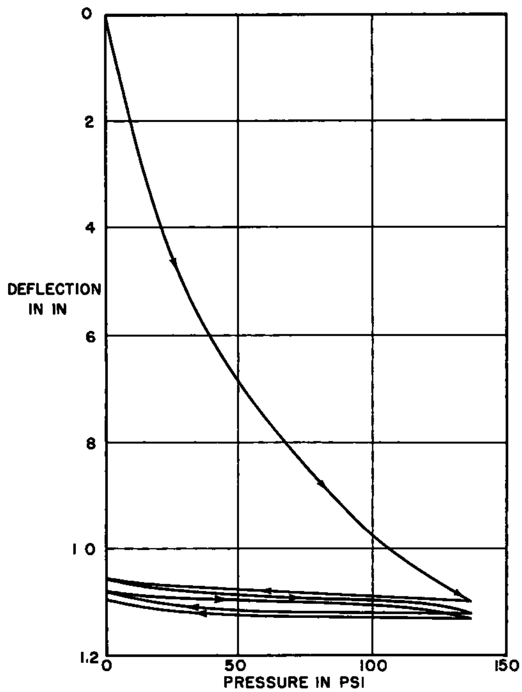


Figure 2. Pressure-deflection curve for three load applications.

tinuing on this point at Georgia Tech, but the tentative conclusion is that the soil density increases with the square of this ratio, until the ratio equals approximately 1.

The most-important factor was found to be the percentage of the total energy which was applied in each tamp, blow, or application of energy to the soil. The greatest density in every case was produced when all the energy was utilized in a single application; the greater number of applications required to apply the same amount of energy, the smaller will be the resulting density. A typical curve from these tests is given in Figure 1.

CONSOLIDATION AND COMPACTION

The cause of this reduction in compaction effectiveness as the number of applications of the compaction energy increases must lie in the mechanics of the compaction process. Additional research was therefore directed toward that end.

Compaction of cohesive soils is essentially consolidation with limited lateral support. This is produced by pressure regardless of whether the soil is tamped, hammered, or just loaded statically. When increasing pressure is applied to a soil it deforms or compacts. When the pressure is released, the soil swells but not to its original volume. If the same pressure is reapplied, additional consolidation will take place, but not nearly as much as during the first application. The rebound will be proportionally greater. Each successive application of pressure will produce less and less consolidation and proportionally more and more rebound until the two are equal and no further consolidation occurs. The pressure-deflection curves for three successive applications of a 140-psi. pressure to a 3-sq.-in. compaction foot acting on a 2-inch soil layer are shown in Figure 2.

The amount of work exerted in each cycle of pressure application and release can be found by integrating the pressure deflection curve. From Figure 2 it can be seen that the amount of work exerted is greatest during the first pressure application and much less during each successive application. Figure 3 shows how both the amount of compaction and the amount of work decrease sharply with successive load applications.

The effectiveness of the compaction can be expressed by the compaction ratio—the ratio of the amount of compaction to the work done in producing it. Figure 3 shows that it becomes less with each successive pressure application until it probably eventually becomes zero. In other words, the first application of pressure produces by far the most compaction for the amount of work required, and is therefore the most efficient.

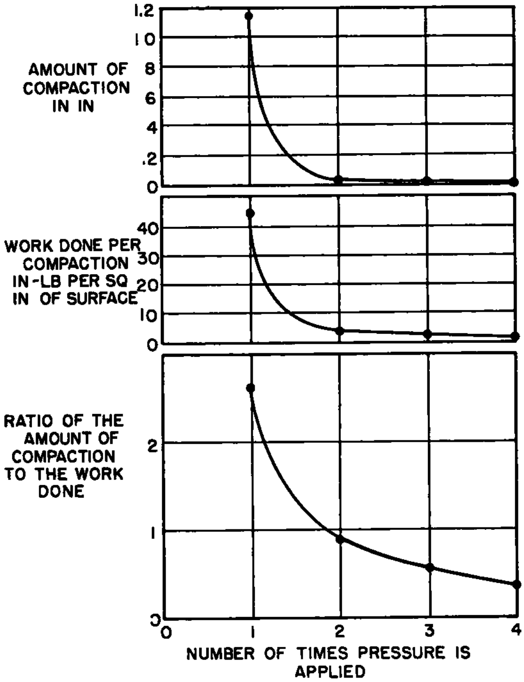


Figure 3. The amount of compaction, the work done and the compaction ratio (ratio of compaction to work) as functions of the number of pressure applications.

The cause can be inferred from our knowledge of soil structure. The compaction of the soil under pressure is the result of elastic deflection of the soil structure and plastic movement of the soil grains into a more dense arrangement. The elastic deflection absorbs work in the form of strain energy while the plastic deformation absorbs work and transforms it to heat. When the pressure is removed, the elastic part of the deflection is largely recovered. The strain energy is dissipated in the viscous resistance of the soil to swelling and in some plastic deformation and re-arrangement. During a second load application the deflection is largely elastic with only a small amount of plastic deformation because the grains were already readjusted during the first loading. The only reason additional compaction is produced is that some readjustment took place during swelling. After many load applications the deflection is entirely elastic and no additional grain adjustment takes place. Work is still exerted, however, to overcome the viscous resistance to deformation although no additional compaction results from it.

SUMMARY OF CONCLUSIONS

The results of this research lead to three conclusions which have important implications in field and laboratory practice: (1) Each successive application of the same pressure to the soil results in less and less work done per application. (2) Each successive application of pressure results in less compaction per unit of work. (3) For a given amount of work, the greatest compaction results when the work is exerted in a single application.

APPLICATIONS TO FIELD AND LABORATORY COMPACTION

When the action of modern compaction equipment is evaluated with respect to the above conclusions, some devices are seen to be inefficient. The sheepsfoot roller, for example, requires theoretically from 10 to 15 passes to secure complete coverage of an area. Actually with random rolling many parts of an area are rolled two or three times and other parts not at all. A roller designed with more and larger feet would produce less overlapping and better compaction for the amount of work done. The same applies to rubber-tired rollers. For greatest efficiency these should have tire spacings and arrangements so that all parts of the surface can be covered just once without appreciable overlap.

Specifications should be written to require just one complete surface coverage. If sufficient density is not obtained in a single coverage (and the soil moisture is correct) the equipment is inadequate, and additional rolling with the same equipment would be largely a waste of time and money. This is particularly important when the owner must pay for all passes of the roller above a certain minimum.

The difference between the moisture-density curves developed in the laboratory by the customary 25 blows of a hammer on layers of soil in a 4-inch-diameter mold and moisture-density curves developed by rolling can be easily explained by these studies. In the laboratory test the large number of blows results in an average of six applications of pressure to the soil. In contrast, even 10 passes of a sheepsfoot roller (a large number) may not even produce one complete coverage of the surface. Furthermore the constant pressure exerted by the sheepsfoot roller means a decreasing amount of work for each pass of the roller while the laboratory compaction hammer exerts an equal amount of energy each time. The remedy would be to use a much heavier hammer for laboratory compaction and fewer blows, say six per layer. On this basis the author believes that a laboratory compaction test can be developed which will retain the simplicity of the present standard Proctor and yet be similar in its results to field experience.

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