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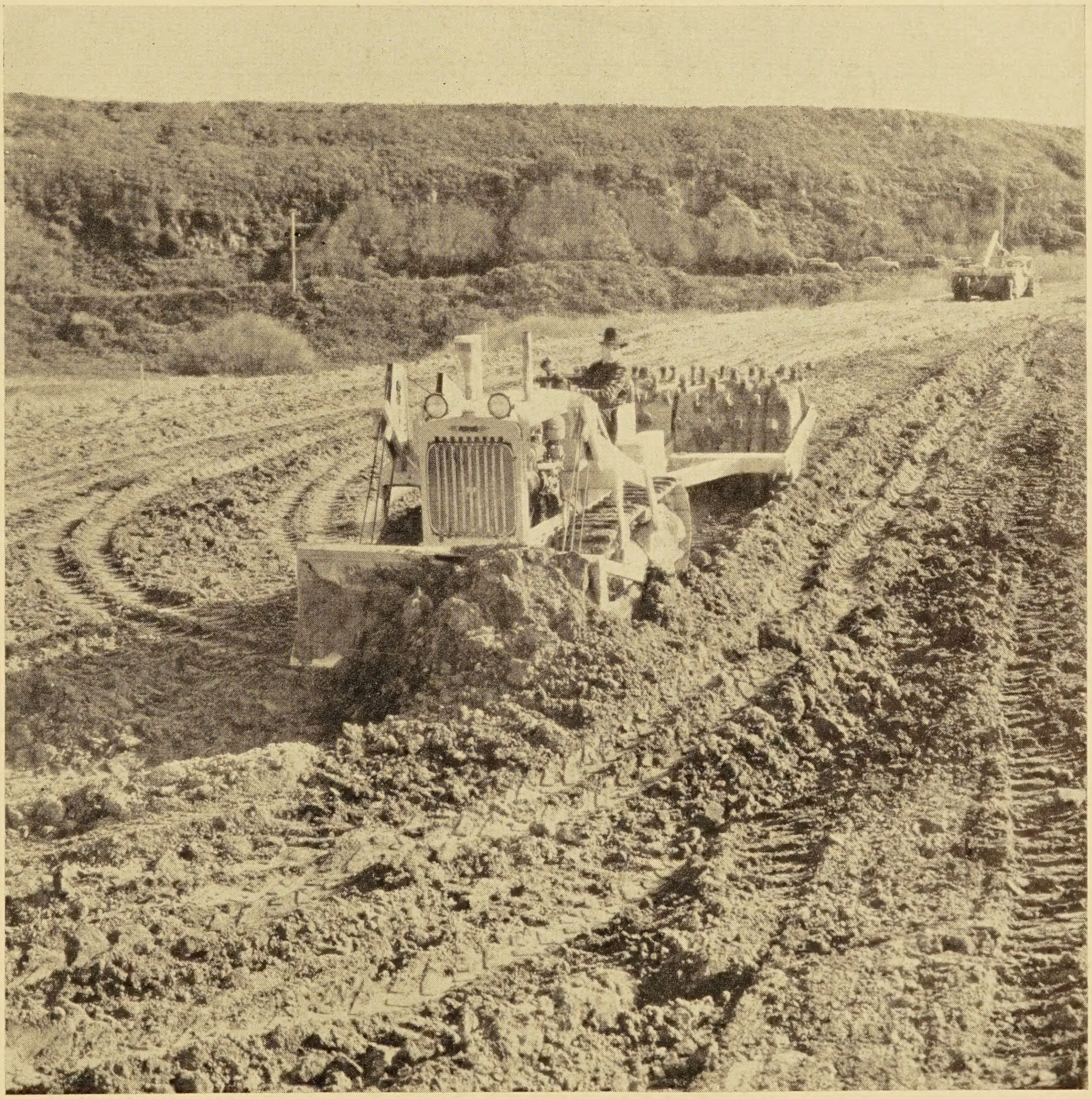

SPREADING AND COMPACTING SOIL IN A HIGHWAY FILL

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

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D. M. BEACH, *Editor*

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

In This Issue

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CLASSIFICATION OF SOILS AND CONTROL PROCEDURES USED IN CONSTRUCTION OF EMBANKMENTS

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by HAROLD ALLEN, Materials Engineer

Methods of testing soils and the use of the test results in a classification system were presented in the June and July 1931 issues of PUBLIC ROADS. Desirable changes in the system have been developed through wide usage by highway engineers. The revised methods of testing and the simplified classification system reported are based on these developments.

The standard method of test for the determination of the relationship of soil moisture and density is described. The use of the results obtained by this testing procedure in soil classification and in the construction of embankments is discussed.

Construction methods used in the control of water content and compaction of soil are described. Testing procedures designed for field use in checking soil moisture and density are reported.

THE PURPOSE of this report is to present a revised and simplified version of soil identification and classification and to describe the field testing procedures which have been used successfully for the control of the work in the building of embankments. The method of soil grouping and classification, originally devised by the Public Roads Administration, has been widely used throughout the United States. A complete analysis of the original soil grouping and its application was published in the June and July 1931 issues of PUBLIC ROADS.[1]

The test procedures used for the determination of soil characteristics are A. A. S. H. O.[2] and A. S. T. M.[3] standards and complete details may be obtained from the publications of these organizations.

The physical properties of soil and the tests upon which they are based are outlined briefly as follows:

Mechanical analysis (*MA*)	Grain size.
Liquid limit (*LL*)	Plasticity.
Plastic limit (*PL*)	
Plasticity index (*PI*)	
Shrinkage limit (*SL*)	Volume change.
Shrinkage ratio (*SR*)	
Lineal shrinkage (*LS*)	
Field moisture equivalent (*FME*).	Moisture capacity of soils.
Centrifuge moisture equivalent (*CME*).	Resistance to flow of water.

Mechanical analysis.—The mechanical analysis of soils determines the size and grading of the particles. The grain sizes of the particles retained on a No. 200 sieve are determined by sieve analyses. The sizes of the soil particles passing a No. 200 sieve are determined by hydrometer analyses.

The hydrometer method of grain-size analysis is based upon the fact that particles of equal specific gravity settle in water at a rate which is proportional to the size of the particle (Stokes' law).

The hydrometer analysis is made by dispersing an air-dry sample, passing the No. 10 sieve, in water by means of a mechanical disperser such as a milkshake mixer. The soil-water mixture is placed in a liter graduate and water added to increase the volume of the suspension to 1,000 cubic centimeters. The weight of soil in suspension, expressed in grams, is determined by reading a hydrometer (Bouyoucos type) suspended in the soil-water mixture. The readings are taken at intervals of 1, 2, 5, 15, 30, 60, 250, and 1,440 minutes, and are used to calculate the grain size and percentage of each grain size in the sample. The sediment in the test cylinder is washed over a No. 200 sieve after the last hydrometer reading has been taken, dried and sieved with No. 20, 40, 60, and 140 sieves and the accumulative percentages passing each sieve are recorded.

A grain diameter accumulation curve is shown in figure 1.

The results, read from the accumulation curve, are usually reported as follows:

	Percent
Particles larger than 2 millimeters (No. 10 sieve)	
Coarse sand, 2.0 millimeters to 0.25 millimeter (No. 60 sieve)	
Fine sand, 0.25 to 0.05 millimeter (No. 270 sieve)	
Silt, 0.05 to 0.005 millimeter	
Clay, smaller than 0.005 millimeter	
Colloids, smaller than 0.001 millimeter	

All of the soil tests used for identification, except mechanical analysis, are made upon the portion of air-dried soil passing the No. 40 sieve.

SOIL TEST PROCEDURES DESCRIBED

Liquid limit.—The liquid limit is defined as that moisture content, expressed as a percentage by weight of the oven-dry soil, at which the soil will just begin to flow when jarred slightly. According to this definition, soils at the liquid limit have a very small but definite shear resistance which may be overcome by the application of little force. At the liquid limit the cohesion in the soil is practically zero.

The nature of the liquid-limit test is indicated in figure 2. The soil sample is placed in a porcelain evaporating dish about 4½ inches in diameter, shaped into a smooth layer about ⅜ inch thick at the center and divided into two portions by means of a grooving tool of standard dimensions (fig. 3). The dish is held firmly in one hand and tapped lightly 10 times against the heel of the other hand. If the lower edges of the 2 soil portions do not flow together, as shown in the lower part of figure 2, the moisture content is below the liquid limit. If they flow together before 10 blows have been struck, the moisture content is above the liquid limit.

[1] These issues are out of print but can be obtained at many public or college libraries.

[2] Standard Specifications for Highway Materials and Methods of Sampling and Testing, published by the American Association of State Highway Officials, 1220 National Press Building, Washington, D. C.

[3] A. S. T. M. Standards, Part II, published by the American Society for Testing Materials, 260 South Broad Street, Philadelphia, Pa.

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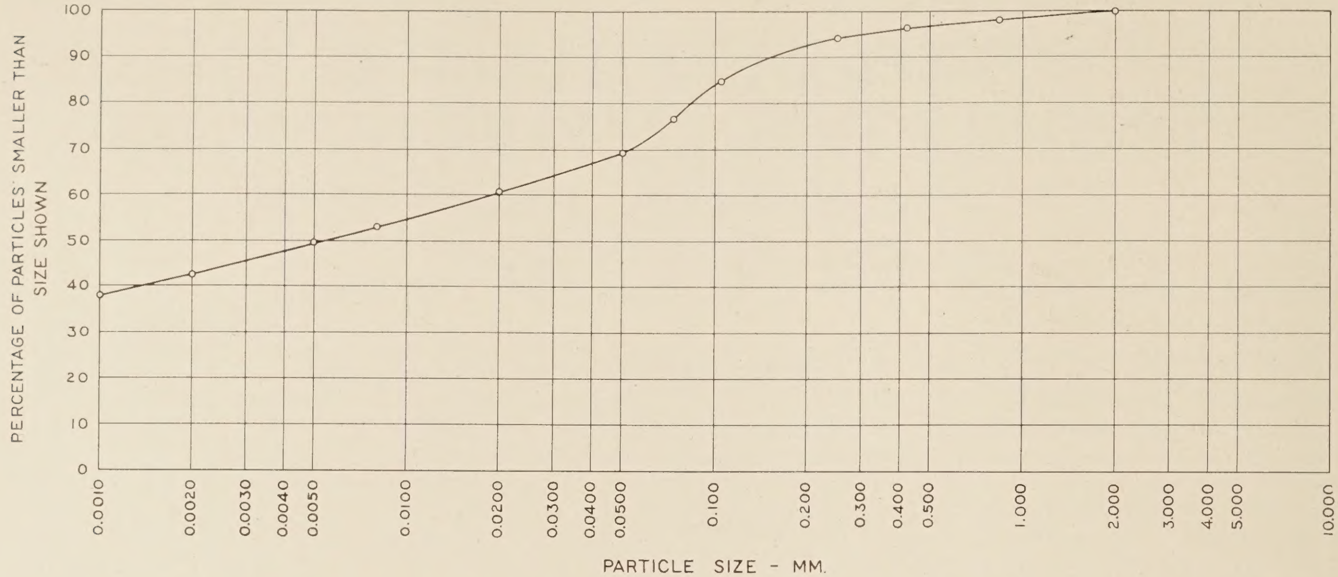


Figure 1.—Grain-Size Accumulation Curve.

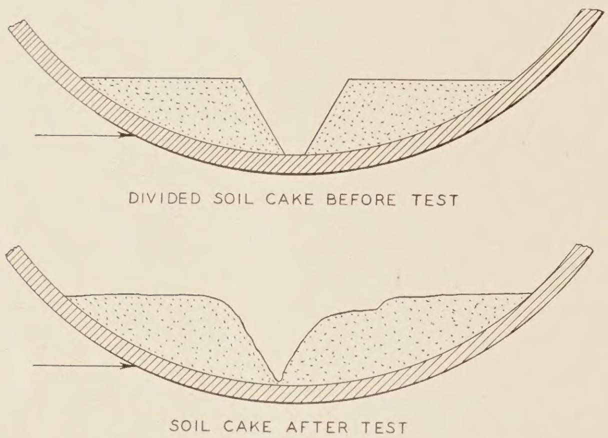


Figure 2.—Phenomenon Occurring During Liquid Limit Test.

The test is repeated with more or less moisture, as the case may be, until the 2 edges meet exactly after 10 blows have been struck. The arrows indicate the direction of the blow on the dish.

A mechanical device which is calibrated against the hand method described above is used in most laboratories. The details of the device are shown in figure 3. In using the device, the soil mixed with water is placed in the brass cup, shaped into a smooth layer, and grooved in a manner similar to that described for the hand method. The cup is then attached to the carriage of the machine and dropped through a distance of 1 centimeter a sufficient number of times to close the groove. This process is repeated for several moisture contents. The object of the procedure is to obtain samples of such consistency that the number of drops or shocks of the cup required to close the groove will be both below and above 25. A "flow curve" is plotted on semilog graph paper using the moisture contents as abscissae on the arithmetic scale and the number of shocks as ordinates on the logarithmic scale. The moisture content corresponding to the intersection of the flow curve with the 25 shock ordinate is the liquid limit of the soil.

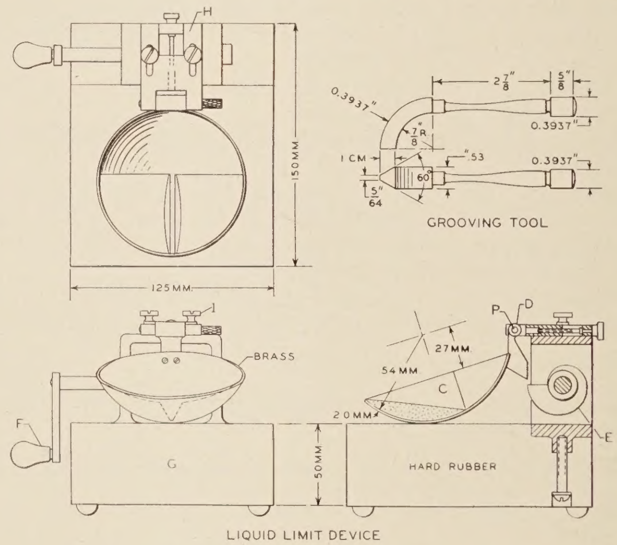


Figure 3.—Liquid Limit Device.

The liquid limits obtained by an operator of average experience and skill, using both methods, should check closely for identical soil samples.

Plastic limit.—The plastic limit is defined as the lowest moisture content, expressed as a percentage by weight of the oven dry soil, at which the soil can be rolled into threads ⅛ inch in diameter without breaking into pieces. Soil which cannot be rolled into threads at any moisture content is considered nonplastic.

Figure 4 shows the nature of the test for the determination of the plastic limit. The sample shown at the top of the figure, having a moisture content above the plastic limit, can be rolled into threads ⅛ inch in diameter without crumbling under the pressure exerted by the hand. The lower part of the drawing shows a soil thread which has crumbled because the moisture content of the soil has been reduced by evaporation to the plastic limit or below.

The plastic limit is the moisture content at which cohesive soils pass from the semisolid to the plastic state. It is also the moisture content at which the

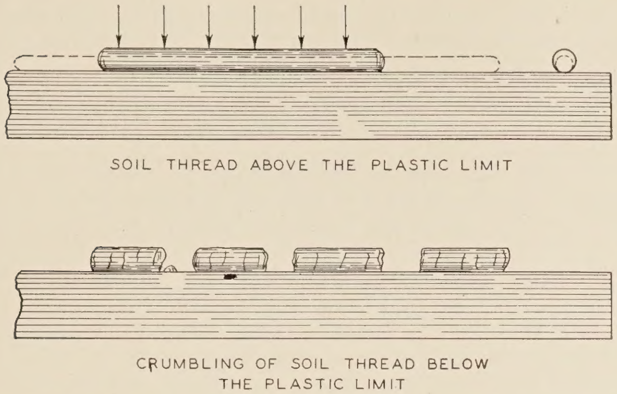


FIGURE 4.—PHENOMENON OCCURRING DURING PLASTIC LIMIT TEST.

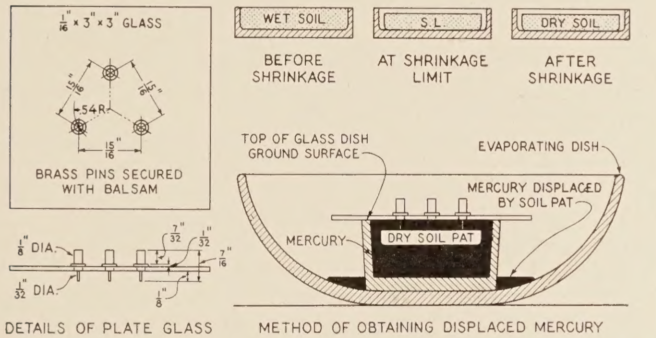


FIGURE 5.—APPARATUS FOR DETERMINING THE VOLUMETRIC CHANGE OF SUBGRADE SOILS.

coefficient of permeability of homogeneous clays becomes practically equal to zero.

Plasticity index.—The plasticity index is defined as the difference between the liquid limit and the plastic limit. It is the range of moisture content through which the soil is plastic. When the plastic limit is equal to or greater than the liquid limit, the plasticity index is reported as zero. When the plastic limit cannot be determined, the plasticity index may be designated by the letters NP (nonplastic) to indicate that the soil is entirely lacking in plasticity.

Shrinkage limit.—The shrinkage limit is defined as the moisture content, expressed as a percentage by weight of oven-dried soil, at which a reduction in moisture content will not cause a decrease in volume of the soil mass, but at which an increase in moisture content will cause an increase in volume of the soil mass. The relations of soil volumes to moisture contents at various stages in the test are illustrated in figure 5.

The shrinkage limit is a means of describing the pore space present in a soil after it has been allowed to compact itself to the maximum density obtainable (from a given moisture content) by shrinkage. It is a well defined point on the moisture content scale, marking the change from the solid to the semisolid state.

Shrinkage ratio.—The shrinkage ratio is equal to the bulk specific gravity of the dried soil pat used in obtaining the shrinkage limit. It is used in the calculation of volume change.

The volume change of soil from a given moisture content can be calculated, when the shrinkage limit and the shrinkage ratio are known, by means of the following formula:

$$VC=(w-S)R$$

in which VC=volume change;
w=moisture content;
S=shrinkage limit; and
R=shrinkage ratio.

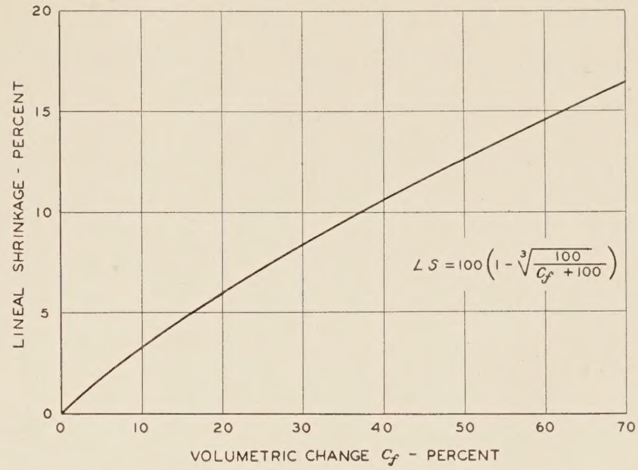


FIGURE 6.—RELATION BETWEEN VOLUME CHANGE AND LINEAL SHRINKAGE.

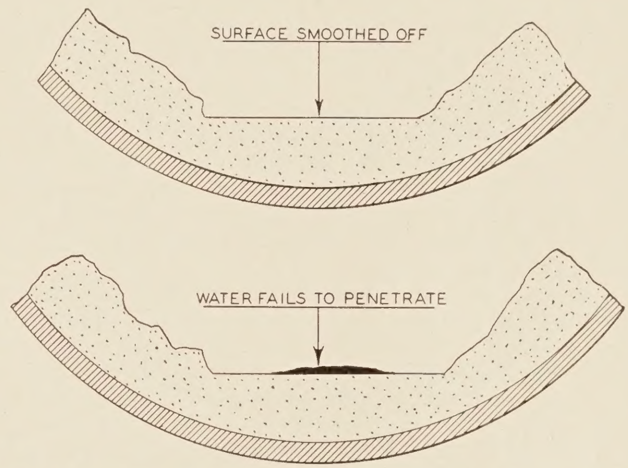


FIGURE 7.—PHENOMENON OCCURRING DURING THE FIELD MOISTURE EQUIVALENT TEST.

The most common value for w is the moisture content represented by the field moisture equivalent (FME) and, using this value, the formula is usually expressed as

$$C_f=(FME-S)R$$

in which C_f is the volume change from the field moisture equivalent.

Lineal shrinkage.—The lineal shrinkage of a soil is the decrease in a dimension of the soil mass, expressed as a percentage of the original dimension, when the moisture content is reduced from an amount equal to the field moisture equivalent to the shrinkage limit. It is usually obtained by calculation by means of the following formula:

$$LS=100\left(1-\sqrt[3]{\frac{100}{C_f+100}}\right)$$

or from the curve of figure 6.

Field moisture equivalent.—The field moisture equivalent is defined as the minimum moisture content, expressed as a percentage by weight of oven-dry soil, at which a drop of water placed on the smooth surface of the soil will not immediately be absorbed but will

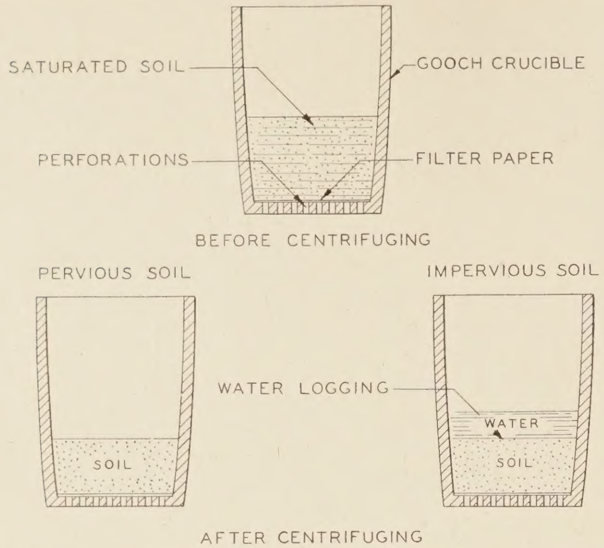


FIGURE 8.—PHENOMENON OCCURRING DURING THE CENTRIFUGE MOISTURE EQUIVALENT TEST.

spread out over the surface and give it a shiny appearance. In making the test, water is mixed with the soil fraction passing the No. 40 sieve until the soil forms into balls when stirred and then in small increments until the moisture content is such that a drop of water will not penetrate the smoothed surface. This is illustrated in figure 7.

The drop of water fails to penetrate the wet and smoothed soil sample (1) when the pores of nonexpansive soils are completely filled, (2) when the capillarity of cohesionless expansive soils is completely satisfied, and (3) when cohesive soils possess moisture in amount sufficient to cause the smoothed surface of the sample to become impervious. This impervious skin may occur at moisture contents far below those required to satisfy the capillarity of cohesive soils.

Centrifuge moisture equivalent.—The centrifuge moisture equivalent is defined as the moisture content, expressed as a percentage by weight of oven-dried soil, retained by a soil which has first been saturated with water and then subjected to a force equal to 1,000 times the force of gravity for 1 hour. The test consists of first soaking a small sample of air-dried soil with water in a Gooch crucible, then draining it in a humidifier for at least 12 hours and, finally, centrifuging it for 1 hour. The effect of the centrifugal force on the soil moisture is illustrated in figure 8.

SOILS CLASSIFIED IN EIGHT GROUPS

Based upon their field performance, soils have been classified in eight groups designated as A–1 to A–8, inclusive. The results of tests made in accordance with the procedures described indicate the physical properties of soils and serve to identify them with respect to grouping. This method of classification does not eliminate possible overlapping or provide a rigid measure of soil behavior. Thus, some soils may have some of the characteristics of two groups. The engineer should learn to judge the value that different soils may have in construction, and the difficulties which may arise in their use, more upon the basis of the physical constants and their relationship than upon the fact that the soils fall in certain groups. This is illustrated by the fact that clay soils from different locations classed in the A–6 or A–7 group may have a wide range of plasticity constants and, therefore, may have different values for fill and subgrade construction. The soil classification should be used to designate general characteristics such as plasticity, permeability, bearing power, resistance to frost heave, etc.

It would be difficult to show all the soil constants in general reports or on soil maps, but the use of the eight groups gives the engineer who is not concerned with details a general picture of the soils on a project. The design and construction engineers, however, should have at their disposal the laboratory test results and should depend more upon those results in preparing specifications and plans and in placing the soils in the finished structure than upon the group classification.

Present knowledge of soil testing indicates a need for slight modification of the classification procedure as originally presented in the June and July 1931 issues of PUBLIC ROADS. The significant changes listed below are included in the simplified charts, figures 9, 10, and 11 which show the range of soil characteristics for each soil group.

1. The relations of the plasticity index to the liquid limit (see fig. 9) have been modified as follows:

a. A band instead of a single curve has been provided to define the limits of the A–6 group.

b. Keeping the origin at a value of the liquid limit equals 14 and a plasticity index equals 0, the curve separating the A–7 group from the A–5 and A–8 groups was rotated to the left slightly. At a value of the liquid limit equals 40, the relation now shows a plasticity index of 15 instead of the value of 16 shown in the original charts.

2. The minimum value of the liquid limit of the A–8 group is given as 35 instead of 45 as originally shown.

3. The maximum value of the liquid limit of the A–1 group was raised from 25 to 35 so as to include stabilized road surface materials covered by the standard A. A. S. H. O. and A. S. T. M. Specifications.

4. The symbol NP has been used for those materials for which the plastic and liquid limits cannot be obtained due to a lack of plasticity in the soil.

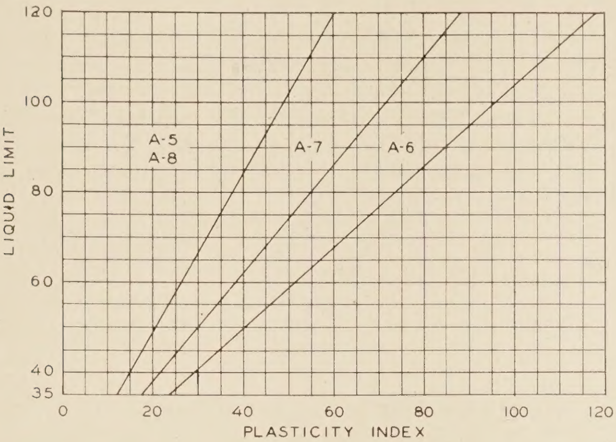
5. The liquid limit values for the A–3 group have been eliminated because the standard test procedure cannot be used on purely granular materials. As a substitute for the effective size of not less than 0.10 millimeter, 0 to 10 percent passing the No. 200 sieve has been inserted.

6. The limiting values for the centrifuge moisture equivalent for all groups except A–1, A–2, and A–3 have been omitted because experience indicates that the test values obtained are not essential for the identification of the remaining groups.

The gradings of the various soil groups, the limits within which the test values fall, and their general characteristics are outlined in the following paragraphs:

GENERAL CHARACTERISTICS OF FIRST THREE SOIL GROUPS GIVEN

Group A–1.—Soils of this group are composed of material well graded from coarse to fine, mixed with excellent binder; they are highly stable under wheel loads irrespective of moisture conditions; can be rolled to very high densities with either smooth-faced or tamping type rollers; and have practically no volume change. These materials have very high bearing capacity at high densities and function satisfactorily when used as bases for relatively thin wearing courses.

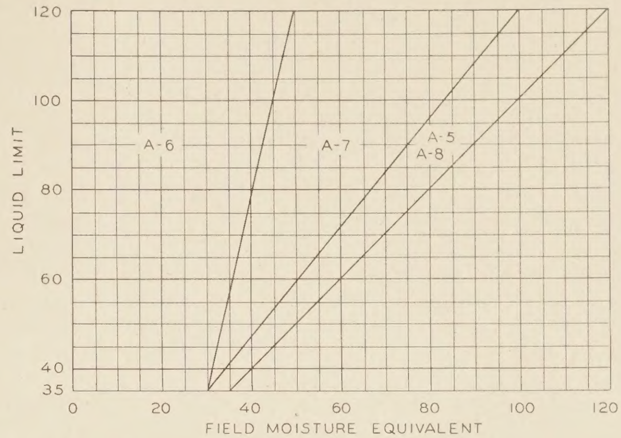


GROUP	A-1	A-2	A-3	A-4	A-5	A-6	A-7	A-8
COARSE MATERIAL	0-65	-o-	-o-	-o-	-o-		-o-	
SOIL MORTAR TOTAL SAND	70-85	55 MIN.	-o-	55 MAXIMUM				
COARSE SAND	45-60	-o-	-o-	-o-	-o-		-o-	
SILT	10-20	-o-	-o-	-o-	-o-		-o-	
CLAY	5-10	-o-	-o-	-o-	-o-		-o-	
PERCENTAGE PASSING NO. 200 SIEVE	-o-	-o-	0-10	-o-	-o-		-o-	
LIQUID LIMIT	14-35	35 MAX.	N P	20-40	35 MINIMUM			
PLASTICITY INDEX	4-9	NP-15	N P	0-15	SEE CHART ABOVE			

NOTE:- ONLY THE A-1 MATERIALS WITH VALUES OF LIQUID LIMIT NOT GREATER THAN 25 AND VALUES OF PLASTICITY INDEX NOT GREATER THAN 6 ARE SUITABLE FOR USE IN BASE COURSES FOR THIN FLEXIBLE SURFACES.

-o- NOT ESSENTIAL NP-NONPLASTIC

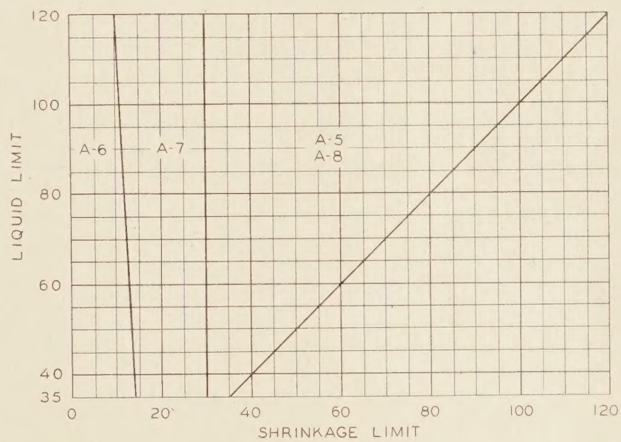
FIGURE 9.—RANGE OF SOIL CHARACTERISTICS FOR EACH SOIL GROUP.



GROUP	A-1	A-2	A-3	A-4	A-5	A-6	A-7	A-8
LIQUID LIMIT	14-35	35 MAX.	N P	20-40	35 MINIMUM			
FIELD MOISTURE EQUIVALENT	-o-	-o-	-o-	30 MAX.	SEE CHART ABOVE			
CENTRIFUGE MOISTURE EQUIVALENT	15 MAX.	25 MAX.	12 MAX.	-o-	-o-			

-o- NOT ESSENTIAL NP-NONPLASTIC

FIGURE 10.—RANGE OF SOIL CHARACTERISTICS FOR EACH SOIL GROUP.



GROUP	A-1	A-2	A-3	A-4	A-5	A-6	A-7	A-8
LIQUID LIMIT	14-35	35 MAX.	N P	20-40	35 MINIMUM			
SHRINKAGE LIMIT	14-20	-o-	-o-	20-30	SEE CHART ABOVE			

-o- NOT ESSENTIAL NP-NONPLASTIC

FIGURE 11.—RANGE OF SOIL CHARACTERISTICS FOR EACH SOIL GROUP.

Grading: The soil mortar, that fraction passing the No. 10 sieve, should be graded as follows: Clay, 5 to 10 percent; silt, 10 to 20 percent; total sand, 70 to 85 percent; coarse sand, 45 to 60 percent.

Constants: The liquid limit is usually greater than 14 and less than 35; the plasticity index ranges from 4 to 9; the shrinkage limit from 14 to 20; the centrifuge moisture equivalent is less than 15. The field moisture equivalent is not a significant test for this type of soil.

The characteristics of this group of soils are such that the test constants fall into a rather narrow band inasmuch as small variations in grading and binder characteristics result in a soil of the A–2 group. Soils in the A–1 group do not exist over widespread areas and are usually found in relatively small deposits. When available in adequate amounts for proper thicknesses, these soils can be used as a base course for bituminous surfaces when the plasticity index does not exceed 6. They are excellent for use as blanketing materials over dry or silt soils.

Group A–2.—The soils of this group are composed of coarse and fine materials mixed with binder but are inferior to the A–1 soils due to poor grading, inferior binder, or both. A–2 materials can be compacted with either tamping or smooth-faced rollers, the density obtainable depending upon the amount, grading, and character of the binder. In road surfaces, A–2 materials may be highly stable when fairly dry or, depending upon the amount and character of the binder, may soften during wet weather or become loose and dusty in dry periods. If used as base courses, plastic soils of this group may lose stability due to capillary saturation or lack of drainage. Some may be damaged by frost.

Grading: The sand content is not less than 55 percent.

Constants: The liquid limit is usually less than 35. Plasticity index may vary from NP to 15 depending upon grading and character of binder. The shrinkage limit usually does not exceed 25 and is significant only when the grading and character of the binder are considered. The centrifuge moisture equivalent does not exceed 25.

Soils falling in this group are of quite common occurrence. The group is usually divided into two parts, namely, the plastic and friable types. The friable type usually has a plasticity index ranging from NP to less than 3 and can be used as base course material for bituminous surfaces where a moisture content sufficient to insure stability can be maintained or where the material is completely confined. This type is also suitable for use as a blanketing material for very plastic subgrades over which concrete pavement is to be placed. The plasticity index of the plastic type ranges from 3 to 15. When the plasticity index of this type of soil exceeds 6, it is not suitable for use as a base for light bituminous surfacing and may cause warping of concrete pavements if large fluctuations of moisture content are likely to occur.

Soils of this group (plastic or friable) may be considered as stable if well compacted and they are satisfactory for the construction of embankments or the blanketing of the plastic or silty soils. They can be drained and may have sufficient plasticity to cause detrimental volume changes. Bituminous materials, portland cement, and other admixtures can be mixed with soils of this group with comparative ease.

Group A–3.—The soils of this group are composed entirely of coarse materials such as sand and gravel; they lack stability under wheel loads except when damp; are only slightly affected by moisture conditions; have no volume change. They cannot be compacted by rolling, but in most instances may be settled by disking and ponding. They drain rapidly and, when adequately confined, make suitable subgrades for all types of pavement.

Grading: The fraction passing the No. 200 sieve is less than 10 percent.

Constants: Soils of this group have no plasticity. The shrinkage limit and field moisture equivalent are not significant. The centrifuge moisture equivalent does not exceed 12.

A–3 soils are of common occurrence. Many of them can be stabilized successfully with bituminous materials.

SOILS OF FOURTH GROUP SUBJECT TO FROST HEAVE

Group A–4.—This group consists predominantly of silt soils containing only moderate to small amounts of coarse material and only small amounts of sticky colloidal clay. When fairly dry or damp, A–4 soils present a firm riding surface which rebounds but little upon removal of load. When water is absorbed rapidly, they may expand detrimentally or lose stability even in the absence of manipulation. They are subject to frost heave.

The soils of this group vary widely in textural composition and range from the sandy loams to silt and clay loams. A comparison of the grain-size analysis curves indicates wide variation in grading within the group.

The sandy loams can be rolled to comparatively high densities with either tamping or smooth-faced rollers and have good stability through a wider range of densities than do the silts and silt loams. They have only small volume change and do not produce severe pavement warping even though compacted in the dry state.

The silt loams and silts cannot be rolled to high densities because of the excess of voids which results from inferior grading and because of a lack of binder material. They are relatively unstable at all moisture contents but especially at the higher moisture contents when they have very low stability (low bearing capacity). Silts and some silt loams are difficult to roll because best rolling results may be obtained only through a very narrow range of moisture. Uniform compaction can be obtained on these soils by the use of smooth-faced rollers, provided the soil is neither too wet nor too dry. If the moisture content is too high or too low, "bridging" will occur with heavy smooth-faced rollers (soils will bulge up ahead and behind the roller) resulting in nonuniform compaction.

The clay loams of this group are somewhat better graded than are the silts and can be rolled to higher densities. On heavy clay loams tamping rollers have proved more effective than rollers of the smooth-faced type. The clay loams are quite stable at the lower moisture contents and higher densities but under these conditions are likely to show detrimental volume change if the moisture content is increased.

Grading: The sand content is less than 55 percent.

Constants: The liquid limit of soils in this group varies from 20 for sandy loams to 40 for clay loams. The plasticity index varies from 0 for coarse silts with no binder to 15 for clay loams. The shrinkage limit varies from 20 for the better graded sandy clay loams with good binder to 30 for silts. The centrifuge moisture equivalent (not essential for classification) varies from 12 to 50, depending upon the porosity and permeability of the soil. The field moisture equivalent does not exceed 30. When the centrifuge moisture equivalent is greater than the liquid limit, soils in this group are likely to be especially unstable in the presence of water. Group A–4 soils are likely to be highly expansive and approach the A–5 group when the field moisture equivalent exceeds the centrifuge moisture equivalent and when the shrinkage limit is greater than 25. The wide range of soils in this group extends from those which border the A–2 group to those which approach the lower limits of the A–5, A–6, and A–7 groups. The borderline soils are often designated as A–4–2, A–4–5, A–4–6, and A–4–7, indicating that they approach the latter group in characteristics, grading, and values of test constants.

Since the soils in this group are subject to frost heave, they should be covered with granular materials in areas where extremely low temperatures prevail and conditions conducive to frost heave exist. The thickness of cover required to prevent heaving varies from 18 to 48 inches.

When wet, these soils may become elastic and show considerable rebound upon removal of load.

The more plastic types in the group will expand with increases in moisture in sufficient degree to cause warping at the joints in concrete slabs if the soils are placed at moisture contents lower than the optimum.[4] Bituminous surfaces require substantial base courses when placed on subgrades consisting of any of the varieties of this group.

SOILS OF FIFTH AND SIXTH GROUPS NOT SUITABLE AS SUBGRADES FOR THIN, FLEXIBLE-TYPE BASE COURSES

Group A–5.—This group is similar to the A–4 group except that it includes very poorly graded soils which contain materials such as mica and diatoms which are productive of elastic properties and very low stability. Soils of this group are likely to be elastic and to rebound upon removal of load even when dry. Elastic properties of these soils interfere with the proper compaction of flexible-type base courses during construction and with the retention of good bond afterward.

Grading: The sand content is less than 55 percent (exceptions occur).

Constants: The liquid limit is usually greater than 35. The plasticity index usually ranges from 0 to 20 but in some cases may be as high as 60. The shrinkage limit is greater than 30 and less than 120 and usually exceeds 50 for the undesirable soils of the group. Field moisture equivalent varies from 30 to 120.

[4] See p. 270 for definition of optimum moisture content.

The soils in this group are not suitable for use as subgrades for thin stabilized base courses or bituminous surfaces. They are subject to frost heave and should be covered with granular materials when they are encountered in subgrades in areas where extreme freezing conditions prevail. They are usually difficult to compact due to their tendency to rebound upon removal of load. It has also been observed that pavements laid over subgrade soils of this group crack excessively.

Group A–6.—This group is composed of predominately clay soils with moderate to negligible amounts of coarse material. In the stiff or soft plastic state they absorb water only when manipulated. They can be compacted to relatively high densities by the use of heavy rollers and can best be compacted with tamping rollers; have good bearing capacity when compacted to maximum practical density; are compressible and rebound very little upon removal of load; are very expansive and productive of severe warping in concrete slabs if placed sufficiently dry to allow water to be absorbed in large quantities.

Grading: The sand content is less than 55 percent.

Constants: The liquid limit exceeds 35, the plasticity index is greater than 18, the shrinkage limit is less than 14, and the field moisture equivalent is less than 50.

The high plasticity indexes of the soils of this group indicate the very cohesive nature of the binder material (clay and colloids) at the lower moisture contents. The cohesion decreases as the moisture content increases. Therefore, since group A–6 soils do not possess much internal friction, they have low stabilities at the higher moisture contents. Consequently, they are suitable for use in fills and as subgrades only when they can be placed and maintained at a relatively low moisture content.

The very low shrinkage limits are indicative of high volume change. This is because any change in moisture content above the shrinkage limit is productive of a corresponding change in volume, and the range from a given moisture content to the shrinkage limit is greatest in soils of the A–6 group. The high shrinkage ratios, which are equal to the bulk specific gravities of the dried soil pats, show that the capillary pressure exerted as evaporation proceeds is of such intensity as to compress the soil particles in a very compact, dense mass. In the field, group A–6 soils are characterized by the presence of shrinkage cracks on all surfaces exposed to drying.

The value obtained in the centrifuge moisture equivalent test, which is not essential to classification, usually exceeds 25. The high values obtained and the fact that waterlogging often occurs in the test indicate that water moves very slowly through soils of the A–6 group even when under a very considerable head. Thus, these soils will take up water very slowly unless manipulated, and, conversely, once they become wet, they will dry out very slowly. The flow of gravitational water through them is negligible and, consequently, ordinary drainage installations are of little value. It should be emphasized that while the rate of flow of water through group A–6 soils is very slow, the capillary pressure which causes moisture to move from the wetter to the drier portions is very great and large forces can be developed for that reason.

Low field moisture equivalents are characteristic of compressible soils which rebound but little upon the removal of load. In the test a load is applied by means of the spatula which tends to compress and reduce the pore space on the smoothed surface. Particles of an elastic soil tend to separate and so absorb more water and have higher field moisture equivalents than the compressible soils.

Soils of the A–6 group are confined within closer limits in their general characteristics than are those of either the A–4 or A–7 group. Borderline soils are often designated as A–6–4 or A–6–7 soils.

Soils in this group are not suitable for use as subgrades under thin flexible base courses or bituminous surfaces because of the large volume changes that are caused by moisture fluctuations and the loss of bearing power upon the entrance of moisture. When concrete slabs are placed over these soils, the subgrades should be blanketed with nonexpansive materials or should be compacted to high densities at carefully controlled moisture contents. Areas immediately adjacent to the slab which are exposed to drying should be protected by covering with nonexpansive material, such as A–1 or friable A–2 soil, or other insulating material to prevent loss of moisture by evaporation from the subgrade and subsequent warping of the pavement due to reentrance of moisture.

Soils of this group occurring in subgrades for macadam or similar porous base courses should be covered with an impervious, nonexpansive material similar to soil of the A–1 or A–2 groups.

GROUP 7 SOILS TO BE USED WITH CARE; GROUP 8 SOILS TO BE AVOIDED

Group A–7.—Soils of this group are similar to those of the A–6 group except that at certain moisture contents they are elastic and deform quickly under load and rebound appreciably upon removal of load. This characteristic results from an inferior grading (steep grain-size curve through the silt fraction); from extraneous material such as organic matter, mica flakes, lime carbonate; from a variation in grain shapes or from a combination of any two or more of these causes. Alternate wetting and drying of the A–7 soils under field conditions leads to rapid and detrimental volume changes.

The soils of this group are more difficult to compact by rolling than are those of the A–6 group. Heavy tamping rollers have been found most effective for rolling A–7 soils. Soils in this group have good bearing capacities when compacted to high densities but are subject to excessive volume change unless properly compacted at a moisture content sufficiently high to insure minimum air voids. These soils have produced more severe warping of concrete slabs than have soils of other groups.

Grading: The sand content is less than 55 percent.

Constants: The liquid limit for soils of this group exceeds 35 and the plasticity index is greater than 12. The shrinkage limit may vary from 10 to 30; the field moisture equivalent may vary from 30 to 100.

The major difference between soils of the A–7 and A–6 groups is in their elasticity. This property is indicated by the higher shrinkage limit and the higher field moisture equivalent associated with soils of the A–7 group. The higher shrinkage limit may be due to poor grading or poor binder (binder which includes chalk, mica flakes, or an excess of organic matter). Similarly, the field moisture equivalent may be higher due to the higher absorption characteristics of a soil which has poor grading (considerable pore space) or which is made up of the constituents mentioned above.

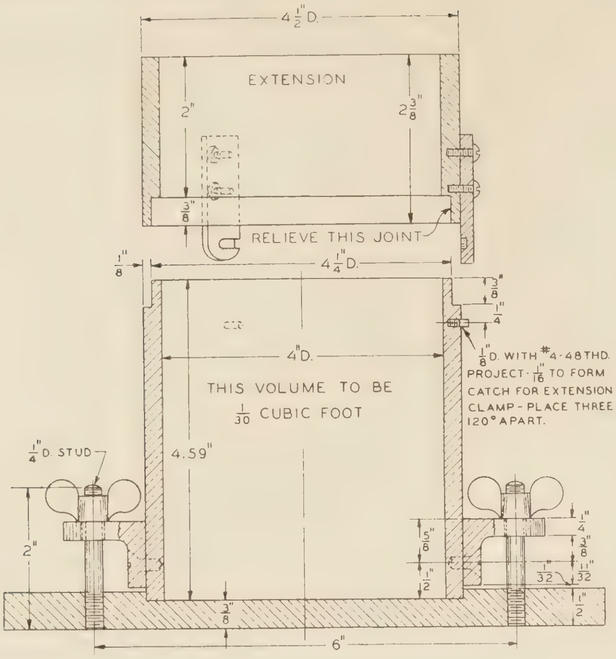


Figure 12.—Compaction Mold.

Like the A–4 group, the A–7 group constitutes a wide range of soils varying in characteristics from those bordering on the A–4 and A–5 groups of silts and loams and the A–6 group of clays to those approaching the lower limits of the A–8 group, which contain excessive organic material. Such borderline soils are often designated as A–7–4, A–7–5, A–7–6, and A–7–8 indicating similarities to the latter groups.

Since the soils in this group are even more expansive than those of the A–6 group, the same precautions in their use should be observed. Due to their elasticity and tendency to rebound, they should be compacted with great care when used as subgrades for concrete slabs, and should not be subjected to excessive loading immediately ahead of paving, if early cracking of the slab due to the force exerted by the rebounding soils is to be avoided.

In areas where low temperatures prevail, the soils in this group should be regarded with suspicion because some of them are subject to frost heave.

Group A–8.—The soils in this group are composed of very soft peat and muck. They contain excessive quantities of organic matter and moisture. They are obviously unsuitable for use in subgrades or embankments.

Grading: The grading is not significant.

Constants: The liquid limit ranges from 35 to 400, the plasticity index from 0 to 60 and is usually less than 25, the shrinkage limit varies from 30 to 120, and the field moisture equivalent from 30 to 400.

The high shrinkage limits and high field moisture equivalents are indicative of the presence of partly decomposed organic matter. The tendency to contain capillary moisture in large amounts far above the water table makes these soils unsatisfactory for use as foundation soils for embankments. Their use in any type of construction should be avoided whenever possible.

In addition to the results of the indicator tests already described, the density-moisture relations of soils in the compacted state are indicative of their value for embankment construction and as foundation materials. Under a fixed set of test conditions each soil has a maximum weight per unit of volume at one moisture content which is known as the optimum moisture content. The maximum dry weight varies with the soil type, being highest for granular well-graded soils in the A–1 group

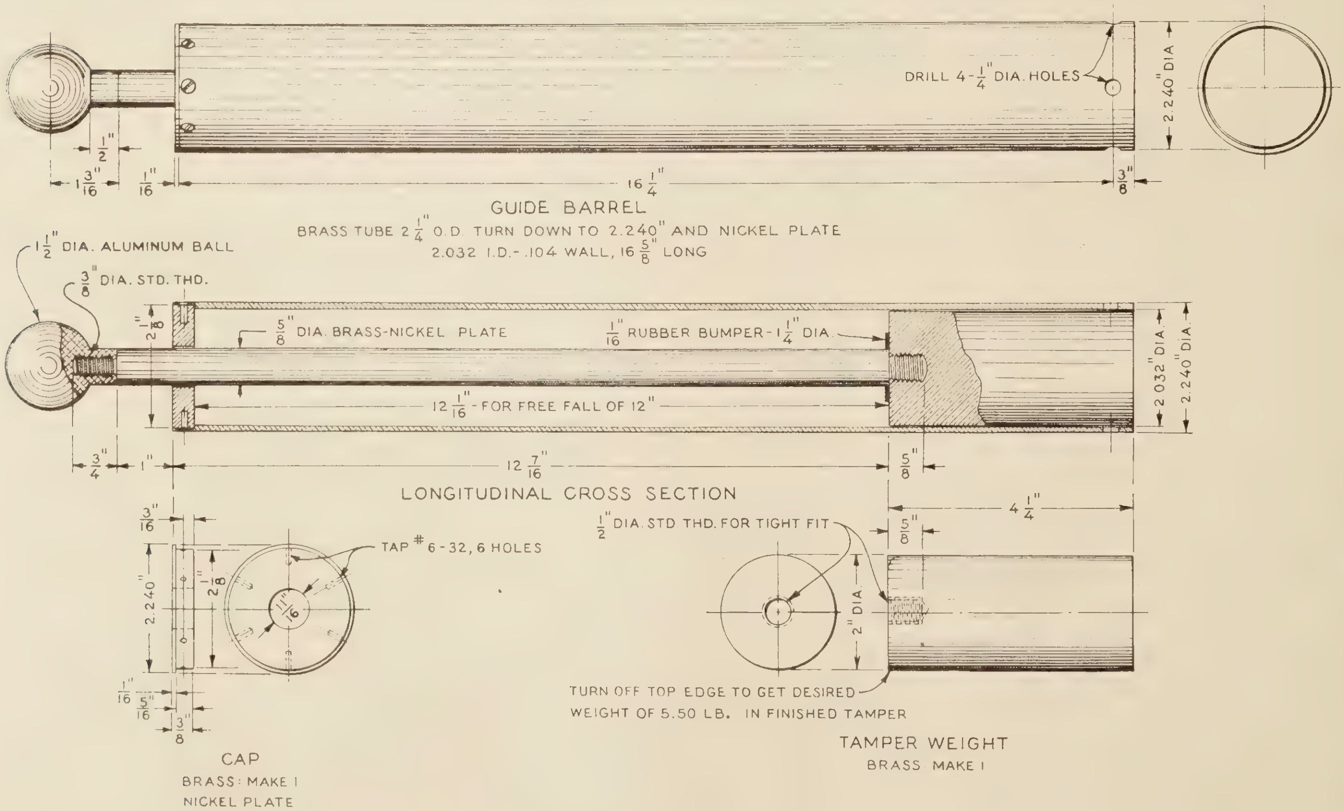


Figure 13.—Soil Tamper.

and decreasing to a minimum for soils in the A–5, A–6, A–7, or A–8 groups. In addition to the relation between density and moisture, a procedure has been developed for obtaining the relation of the moisture content and the resistance to penetration of a needle forced into the compacted soil under fixed conditions.

STANDARD COMPACTION TEST APPARATUS AND PROCEDURE DESCRIBED

The method of test for determination of the moisture-density and moisture-penetration relations is designated as the "standard compaction test" and is conducted in accordance with the following procedure.

The apparatus used shall consist of the following:

1. A cylindrical metal mold approximately 4 inches in diameter and 4½ inches high and having a cubical content of 1/30 cubic foot. This mold is fitted with a detachable base plate and a removable extension approximately 2½ inches high. (See fig. 12.)
2. A metal tamper having a striking face 2 inches in diameter and weighing 5½ pounds. (See fig. 13.)
3. A steel straightedge about 10 inches long.
4. A penetrometer to register the force required to cause the penetration of needles of known end area. (See fig. 14.)
5. A scale of 30 pounds capacity sensitive to ½ ounce.
6. A balance of 100 grams capacity sensitive to 0.1 gram.
7. Porcelain evaporating dishes.
8. Oven for drying soil samples.

The procedure is as follows:

A 6-pound sample, air dried to slightly damp, is taken from a portion of the material passing the No. 4 sieve.

The sample is thoroughly mixed and then compacted in the cylinder (with the extension attached) in three equal layers, each layer receiving 25 blows from the tamper dropped from a height of 1 foot above the soil. The extension is then removed. The compacted soil is carefully leveled off to the top of the cylinder with the straightedge and weighed. The weight of the compacted sample and cylinder, minus the weight of the cylinder, is multiplied by 30 and the result recorded as the wet weight per cubic foot of the compacted soil.

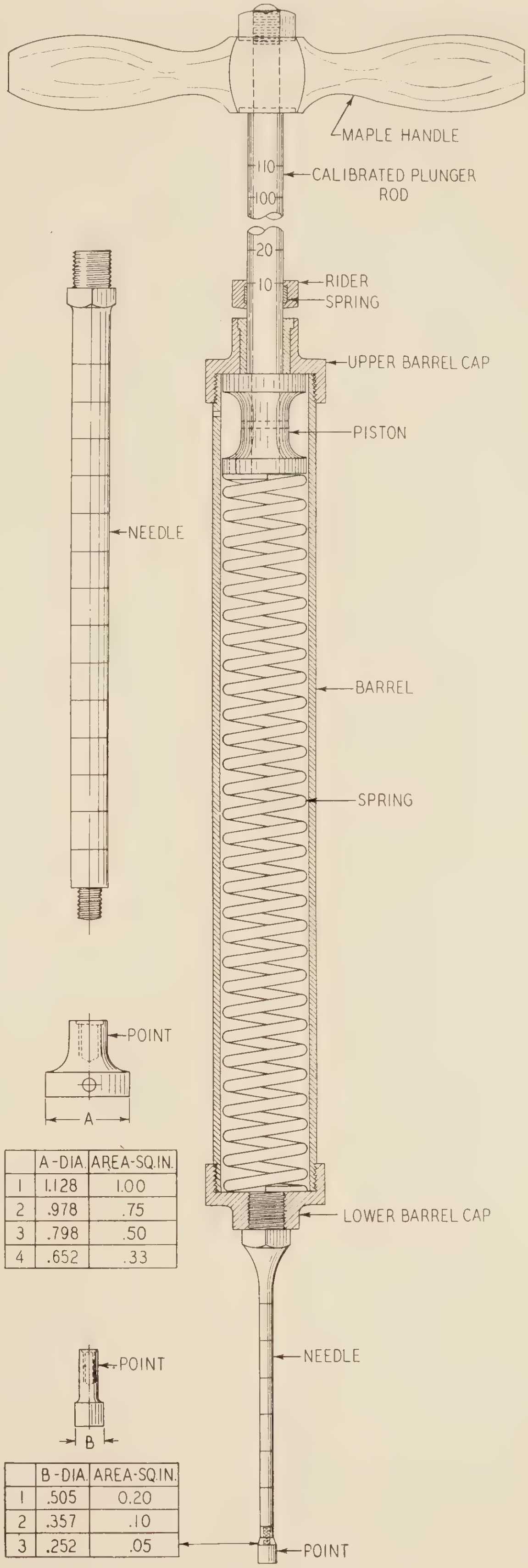
The compacted sample is tested with the penetrometer (fig. 14) and the resistance to forcing the needle into the soil at the rate of ½ inch per second to a depth of 3 inches is recorded. When the material is granular enough to interfere with the uniform penetration of the needle, the penetrometer test cannot be made.

A small sample of the compacted soil is oven dried to determine the moisture content.

The soil is removed from the cylinder and broken up until it will pass a No. 4 sieve. Water in sufficient amounts to increase the moisture content of the soil sample by increments of approximately 1 percent is added and the above procedure repeated for each increment of water added. This series of determinations is continued until the soil becomes very wet and there is a substantial decrease in the wet weight of the compacted soil.

The moisture content (percent by weight of dried soil) of the oven-dried sample is computed from the formula

$$100\times\frac{\text{weight of dish and wet soil}-\text{weight of dish and dried soil}}{\text{weight of dish and dried soil}-\text{weight of dish}}$$



	A-DIA.	AREA-SQ.IN.
1	1.128	1.00
2	.978	.75
3	.798	.50
4	.652	.33

	B-DIA.	AREA-SQ.IN.
1	.505	0.20
2	.357	.10
3	.252	.05

FIGURE 14.—SOIL PENETROMETER.

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The dry weight per cubic foot of compacted soil is computed from the formula

$$\frac{\text{wet weight per cubic foot}}{1+\frac{\text{percent moisture}}{100}}$$

MOISTURE-DENSITY CURVES USEFUL

Curves showing the relations of the moisture contents to the wet and dry densities of the compacted soil, expressed in pounds per cubic foot, and the penetrometer readings, expressed in pounds per square inch, may then be drawn on rectangular coordinate paper to such a scale as to permit reading the moisture contents to 0.2 percent. The peak of the moisture-density curve represents the maximum density for the soil tested and the percentage of water at this point represents the moisture content necessary for maximum compaction. The curves are used in classification and for control during construction.

The above procedure is designed to be used in laboratories where the facilities and time are adequate to permit the breaking down of the soil cylinder for the addition of each increment of moisture. In field laboratories the use of a separate sample for each increment of moisture has proved satisfactory. The samples should be prepared by breaking down approximately 40 pounds of soil from the borrow pit or fill to pass a No. 4 sieve, and drying or adding moisture to make the soil slightly damp. About 5 pounds of the soil thus prepared should be tested in accordance with the procedure described above. The procedure should be repeated by adding enough water so that the moisture content of each successive sample will be about 1 percent greater than the previous one.

The test data for a typical compaction test are shown in table 1. The wet and dry density and penetration curves are shown in figure 15.

The dry weight per cubic foot of soil as determined by the method described above is indicative of the suitability of the material for use in embankments and subgrades. With few exceptions the weight per cubic foot of soil determined by this method varies from 80 to 130 pounds. The granular materials, such as the well graded A–1 or A–2 soils, have the higher weights, and the highly plastic clays or muck soils (A–6, A–7 or A–8) will be at the lower end of the scale.

The Public Roads group classification, the rating for use in embankment construction on the basis of dry weight per cubic foot, the required compaction during construction, and the required thickness of sub-base,

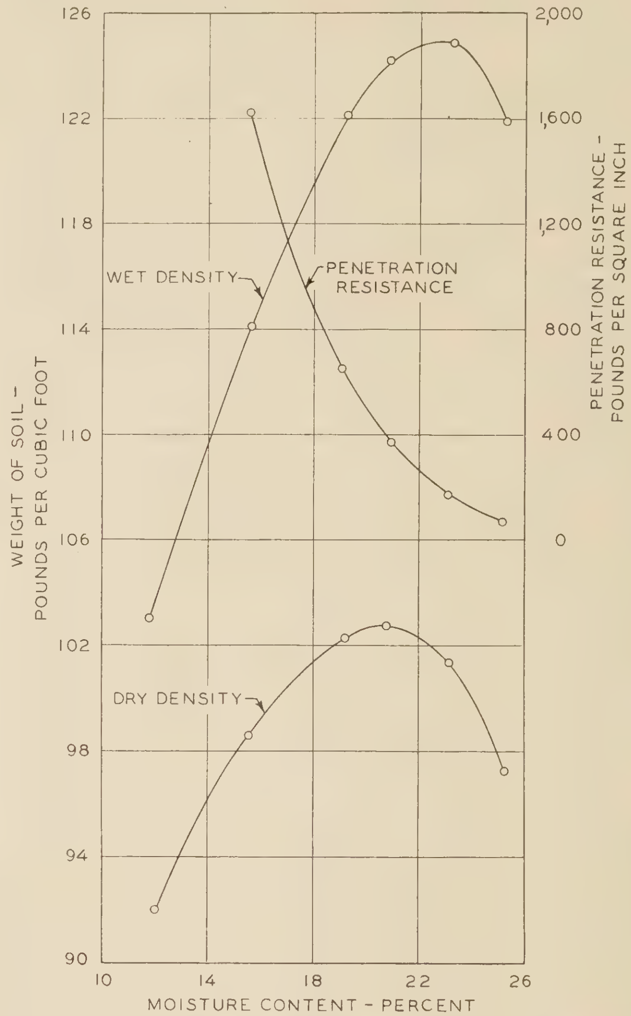


FIGURE 15.—DENSITY AND PENETRATION CURVES.

base, and surfacing, are included in the general summary of soil characteristics and classification shown in table 2.

The approximate grading limits shown in this table will serve as a guide in the visualizing of the textural characteristics of the various soils and, except for those falling in the A–1 group, are not essential to classification.

The required compaction during construction, the procedures for obtaining it, and the methods of testing soil in place will be discussed later.

The rating of soils in table 2 is intended as a guide

TABLE 1.—*Compaction test data*

Weight of compacted sample [1] (pounds)	Wet weight of sample	Penetration test			Moisture determination							Dry weight of soil
		Needle		Pressure	Dish No.	Wet weight	Dry weight	Water weight	Dish weight	Soil weight	Water	
		Size	Reading									
	Pounds per cubic foot	*Square inch*	*Pounds*	*Pounds per square inch*		*Grams*	*Grams*	*Grams*	*Grams*	*Grams*	*Percent*	*Pounds per cubic foot*
3.433	103.0	1/20	[2] 100	[2] 2,000	1	85.08	79.34	5.74	30.72	48.62	11.8	92.1
3.803	114.1	1/20	81	1,620	2	87.47	80.45	7.02	35.55	44.90	15.6	98.7
4.070	122.1	1/20	33	660	3	90.77	81.71	9.06	34.60	47.11	19.2	102.4
4.140	124.2	1/20	19	380	4	89.99	81.46	8.53	40.51	40.95	20.8	102.8
4.161	124.8	1/20	9	180	5	88.53	79.00	9.53	37.96	41.04	23.2	101.3
4.063	121.9	1/20	4	80	6	84.83	75.58	9.25	38.80	36.78	25.2	97.3

[1] Maximum density 102.8 pounds per cubic foot; optimum moisture 20.8 percent.
[2] Greater than capacity of apparatus.

and not as a specification requirement. For example, if a soil weighing 100 to 110 pounds per cubic foot, which is classified as poor or very poor, is the only one available for the construction of an embankment, this classification should be interpreted to mean that the design of the embankment should be given special consideration, and that the soil should be compacted above the minimum requirements during construction.

The curves and data of figures 16 and 17 show the moisture-density and grain-size accumulation curves for typical soils from each of the groups except A–1 and A–8. Curves for samples of two soils classified in the A–2, A–3, A–4, and A–6 groups are shown in order to demonstrate the variation which may exist in soils having the same classification.

THICKNESSES OF SUB-BASE, BASE COURSE, AND SURFACE DEPEND ON SEVERAL FACTORS

Since the results of indicator tests have been correlated with the service behavior of soils in highway construction, it is possible to estimate the required combined thickness of sub-base, base course, and surfacing required for any type of soil. This information is shown in the last line of table 2 and represents the maximum and minimum thickness of sub-base and pavement (base course and surfacing) required for each soil type. These thicknesses were arrived at by observation and not by laboratory or field test or other purely scientific approach. The values, however, are the result of the experience of many engineers concerned with the successful use of soil materials and may be used with confidence.

The combined thickness of the sub-base composed of selected material, base course, and surfacing for each soil type, as shown in table 2, will vary with variations in the soil constants, in degree of compaction obtained, in the climatic conditions, and in the natural soil moisture. For example, a soil of the A–6 group with a plasticity index of 20 and a natural moisture content of 18 percent will require less cover than an A–6 soil with a plasticity index of 50 and a natural moisture content of 30 percent. When used in a dry climate and where the distance to ground water is great, the first soil (plasticity index of 20) will require less cover than where the ground

TABLE 2.—*Summary of soil characteristics and classification*

Group	A-1	A-2		A-3	A-4	A-5	A-6	A-7	A-8
		Friable	Plastic						
General stability properties.	Highly stable at all times.	Stable when dry; may ravel.	Good stable material.	Ideal support when confined.	Satisfactory when dry; loss of stability when wet or by frost action.	Difficult to compact; stability doubtful.	Good stability when properly compacted.	Good stability when properly compacted.	Incapable of support.
Physical constants:									
Internal friction	High	High	High	High	Variable	Variable	Low	Low	Low.
Cohesion	do	Low	do	None	do	Low	High	High	Do.
Shrinkage	Not detrimental.	Not significant.	Detrimental when poorly graded.	Not significant.	do	Variable	Detrimental	Detrimental	Detrimental.
Expansion	None	None	Some	Slight	do	High	High	do	Do.
Capillarity	do	do	do	do	Detrimental	do	do	High	Do.
Elasticity	do	do	do	None	Variable	Detrimental	None	do	Do.
Textural classification:									
General grading	Uniformly graded; coarse-fine excellent binder.	Poor grading; poor binder.	Poor grading; inferior binder.	Coarse material only; no binder.	Fine sand cohesionless silt and friable clay.	Micaceous and diatomaceous.	Deflocculated cohesive clays.	Drainable flocculated clays.	Peat and muck.
Approximate limits:									
Sand ... percent	70-85	55-80	55-80	75-100	55 (maximum)	55 (maximum)	55 (maximum)	55 (maximum)	55 (maximum).
Silt ... do	10-20	0-45	0-45	[1]	High	Medium	Medium	Medium	Not significant
Clay ... do	5-10	0-45	0-45	[1]	Low	Low	30 (minimum)	30 (minimum)	Do.
Physical characteristics:									
Liquid limit	14-35 [2]	35 (maximum).	35 (maximum).	NP [3]	20-40	35 (minimum)	35 (minimum)	35 (minimum)	35-400.
Plasticity index	4-9 [2]	NP-3 [3]	3-15	NP [3]	0-15	0-60	18 (minimum)	12 (minimum)	0-60.
Field moisture equivalent.	Not essential	Not essential	Not essential	Not essential	30 (maximum)	30-120	50 (maximum)	30-100	30-400.
Centrifuge moisture equivalent.	15 (maximum).	12-25	25 (maximum)	12 (maximum).	Not essential	Not essential	Not essential	Not essential	Not essential.
Shrinkage limit	14-20	15-25	25 (maximum)	Not essential	20-30	30-120	6-14	10-30	30-120.
Shrinkage ratio	1.7-1.9	1.7-1.9	1.7-1.9	do	1.5-1.7	0.7-1.5	1.7-2.0	1.7-2.0	0.3-1.4.
Volume change	0-10	0-6	0-16	None	0-16	0-16	17 (minimum)	17 (minimum)	4-200.
Lineal shrinkage	0-3	0-2	0-4	do	0-4	0-4	5 (minimum)	5 (minimum)	1-30.
Compaction characteristics:									
Maximum dry weight, pounds per cubic foot.	130 (minimum).	120-130	120-130	120-130	110-120	80-100	80-110	80-110	90 (maximum).
Optimum moisture, percentage of dry weight (approximate).	9	9-12	9-12	9-12	12-17	22-30	17-28	17-28	
Maximum field compaction required, percentage of maximum dry weight, pounds per cubic foot.	90	90	90	90	95	100	100	100	Waste.
Rating for fills 50 feet or less in height.	Excellent	Good	Good	Good	Good to poor	Poor to very poor.	Fair to poor	Fair to poor	Unsatisfactory.
Rating for fills more than 50 feet in height.	Good	Good to fair	Good to fair	Good to fair	Fair to poor	Very poor	Very poor	Very poor	Do.
Required total thickness for subbase, base and surfacing, inches.	0-6	0-6	2-8	0-6	9-18	9-24	12-24	12-24	

[1] Percentage passing No. 200 sieve, 0 to 10.
[2] When used as a base course for thin flexible surfaces the plasticity index and liquid limit should not exceed 6 and 25, respectively.
[3] NP—nonplastic.

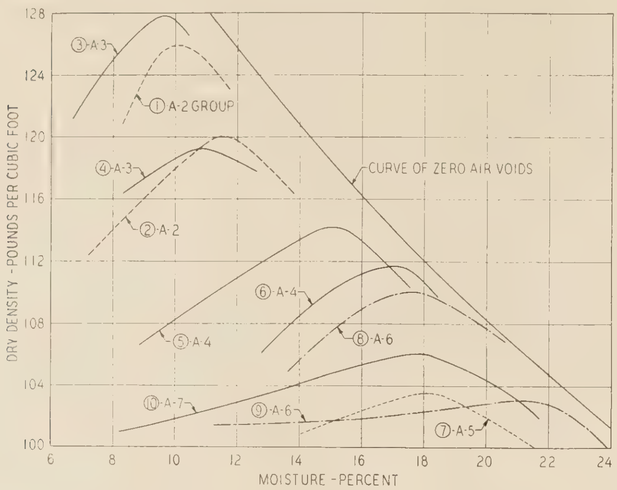


Figure 16.—Moisture-Density Curves.

water is high and the moisture content will be greater throughout the year due to high continuous rainfall. The thickness selected will depend upon the judgment of the engineer.

The selected material for the sub-bases may be composed of soils similar to those of the A–1, A–2, or A–3 groups, natural gravels, which are stable but contain clay of such characteristics or quantity that they are not completely suitable for use in base courses, quarry wastes which are not suitable for base construction, or other materials having low volume change and relatively high density when compacted under a roller.

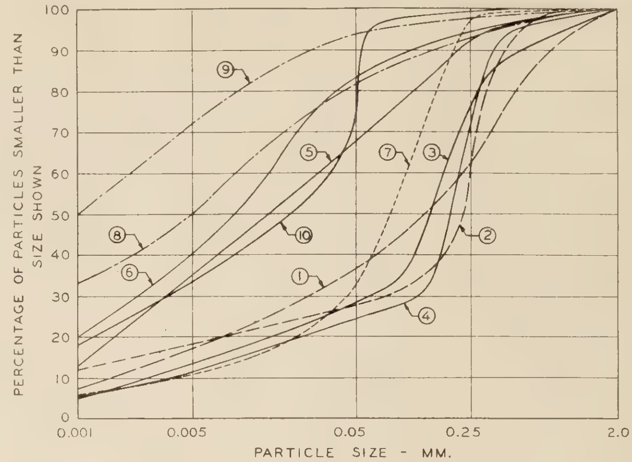
SOIL STABILIZATION EFFECTED BY CAREFUL SELECTION, PLACING, AND ROLLING OF MATERIALS

Properly constructed embankments may be divided on the basis of the method of compaction of the soil material used in their construction into the following types:

1. Uncompacted.
2. Jetted or ponded.
3. Rolled.

The embankments included under the uncompacted classification are those in which the materials consist either of pure sand or of earth mixed with large stones. The latter material usually occurs in mountainous regions or in highly glaciated areas. Since no special compaction methods are necessary to obtain a stable fill with such material, the thickness of lift used in placing the embankment can be much greater than in other types. When sand is used, the method of procedure is governed by the equipment used and is usually worked out to produce the greatest yardage per unit of time. When a mixture of soil and large stones is used, most specifications provide that the material shall be placed in lifts not to exceed 3 feet in thickness and that the fine material be so distributed that no pockets or voids will be left in the finished fill. The equipment used and the methods of procedure in the construction of embankments of this type have been described many times in engineering literature and need not be repeated.

Other embankments in which no special compaction methods are used are those placed with dragline and hydraulic equipment. The soil in this type of work is in a semiliquid state at the time of placing and the resultant fill is uniformly compacted by gravity and drainage to a relatively low density.



PHYSICAL CONSTANTS

	GRADE	SOIL NUMBER	L.L.	P.L.	S.L.	S.R.	C.M.E.	F.M.E.
1	A-2	S 14111	19	2	14	1.9	13	15
2	A-2	S 13908	22	4	—	—	—	—
3	A-3	S 13705	16	0	12	1.9	9	15
4	A-3	S 13703	NP	NP	—	—	5	17
5	A-4	S 13643	28	9	15	1.9	24	20
6	A-4	S 14089	34	13	15	1.8	23	23
7	A-5	S 9043	35	0	29	1.4	17	50
8	A-6	S 9673	54	29	19	1.8	29	25
9	A-6	S 13399	67	40	14	1.9	37	27
10	A-7	S 14135	48	24	16	1.8	30	30

Figure 17.—Grain-Size Accumulation Curves.

Jetting and ponding have been used extensively for the compaction of embankments. Explorations of old fills indicate that this method is successful when soils are sandy and slake down easily when inundated. Heavy clay soils do not compact when jetted and pockets of free water have been found in them several years after completion.

The jetting procedure is somewhat cumbersome and is a separate operation requiring special equipment and attention. The limitations of the method tend to restrict its use to those embankments which cannot be compacted by other means.

The stability of any embankment composed of fine-grained soil is dependent upon the moisture content and the density. There is no single moisture content and density at which soil will remain permanently. There is, however, a moisture content and density at which a soil will offer the greatest resistance to change. An increase or decrease in the moisture content will result in a loss of stability or a change in shape due to shrinkage or expansion. Settlement, softening, shrinkage, swell, and frost heave result from changes in moisture content and changes in temperature. The soil in a structure, therefore, is most stable when it has been placed at a moisture content which offers the greatest resistance to changes in that moisture content. Soil having a moisture content during compaction sufficient to result in a condition of maximum density with the pore spaces as nearly as possible filled with water offers greater resistance to the gain of moisture by absorption or the loss of moisture by evaporation than do soils compacted at any other condition. The process of soil stabilization in embankments consists, therefore, in the introduction of the proper moisture content to obtain a maximum density and the subsequent compaction of the soil mass to that density by means of proper equipment. This condition can best be accomplished by the careful selection, placing, and rolling of soil materials.

The recommended procedure in the construction of rolled embankments is as follows: The soil survey report should be studied by the engineer in charge and soils which are most suitable should be selected for use unless construction limitations make such selection uneconomical. An effort should be made in soil selection to arrange construction procedures so that the most desirable soils will be in the top of the finished grade. It will require close cooperation of the construction and inspection forces to accomplish the distribution of soil materials that will result in the best and most economical soil structure.

Before a plan of construction is adopted, the moisture content of the soil in the various borrow pits should be checked. A study should be made of the moisture-density relations of the soil in the various strata and of the specifications for the project. After these data have been studied, the construction equipment available should be checked over so that the rate at which the work will progress may be determined and any additional equipment necessary for a proper balancing of construction operations may be obtained before work starts. This procedure will also provide the engineer with information from which it will be possible to estimate the number of tests that it will be necessary to make each day and the number of inspectors that will be required to carry on the work most efficiently. The tests made in the field consist chiefly of moisture and density determinations of soils in place either in the borrow pit or in the embankment. A field laboratory should be provided on each project. Such a laboratory usually consists of a portable 10- by 12-foot frame structure properly lighted and equipped with a bench and table for use in making tests and preparing reports. This building is usually placed so that it is convenient to the work and may be moved from time to time as the work progresses.

NECESSARY FIELD LABORATORY EQUIPMENT LISTED

The field laboratory should be equipped with the following:

- 1 compaction mold (fig. 12).
- 1 soil tamper (fig. 13).
- 1 steel straightedge about 10 inches long.
- 1 gasoline camp stove.
- 3 alcohol burning soil moisture apparatuses (figs. 18 and 19).
- 1 small oven with thermometer.
- 1 penetrometer to register the force required to cause the penetration of needles of known end area (fig. 14).
- 1 scale of 30 pounds capacity sensitive to ½ ounce.
- 1 balance of 100 grams capacity sensitive to 0.1 gram.
- 2 4-inch post-hole augers and extensions.
- 1 railroad pick.
- 1 drain spade.
- 12 drying pans.
- 2 6-inch trowels.
- 1 2-gallon can for gasoline.
- 1 8-inch adjustable wrench.
- 1 100 cubic centimeter graduate.
- 1 No. 4 sieve.
- Notebooks, form sheets.
- Miscellaneous articles such as cloth bags, string, etc.

Soil as taken from borrow pits or cuts is usually either too dry or too wet for compaction to maximum density. Therefore, the first operation is preparation of the soil by adjustment of the moisture content.

Soil that is too dry is usually brought to the proper moisture content by irrigation of borrow pits or by sprinkling with water and mixing on the grade with blades, disks, harrows, or other available equipment.

Irrigation may be used either on sidehill locations or on flat areas. When sidehill locations are irrigated, contour ditches are cut with blade graders and water is pumped into the ditches until the desired average moisture content is obtained. On flat areas dikes are constructed and the ponds so formed are kept filled with water until the desired average moisture content is obtained. This method of treatment is suitable on sandy and silty loams which are sufficiently pervious to allow the diffusion of the moisture into the soils in a reasonably short time, but it has not been successful for the treatment of dense, impervious clays. The irrigation method is best adapted for use where heavy embankments are to be constructed from centrally located borrow pits. When rapid penetration is obtained, very little mixing has been found necessary after the material has been deposited on the grade.

Sprinkling may be accomplished by means of hose attached to pipe lines or by the use of gravity sprinkling wagons or pressure distributors. The latter method is the more common. The loose soil is placed on the grade in layers of the thickness necessary to result in the required compacted thickness, the water is added and the mixing done with several types of equipment. Heavy spring-tooth harrows have been used successfully in silty and sandy loams and disk plows have been used in clay loams. Tractor-drawn blades have been found to be most efficient in clay soils of the A–6 and A–7 groups.

The wetting of clay soils to a uniform moisture content is difficult and to be effective must be done very carefully. The following procedure has been found to produce reasonably satisfactory results. The soil is spread in a layer of uniform thickness and sprinkled with water. A shallow cut is made with the blade, placing the wetted soil in a windrow. The operation is repeated until the entire thickness of loose soil has been wetted and placed in the windrow. The wetted windrow is then bladed back into place in thin layers.

When the soil in the borrow pit or cut excavation contains moisture in excess of the optimum, it should be dried until it can be compacted to the density required by the specifications. This may be accomplished to a limited extent with the same equipment and processes which are used in the mixing of moisture into a dry soil. Obviously, such processing cannot begin until the soil has dried sufficiently to permit the working of construction equipment and in many instances further drying may not be necessary. The removal of excess moisture from soil is a much more difficult problem and will require more rigid inspection than the addition of moist ture to dry soil. The process usually results in a delay of the work, but the increase in density and stability of embankments justifies such delay.

ALLOWANCE SHOULD BE MADE FOR EVAPORATION LOSSES

The results obtained in compaction operations will be affected by the placing and spreading of the soil layers. The loads should be so spaced that, when spread, the thickness of the resulting uniform layer will not exceed that necessary to obtain the required density. Soils of the correct moisture content should not be placed and spread so far in advance of rolling operations that they dry appreciably before rolling, since this procedure necessitates the addition of more water, additional mixing, and testing. The loss of some moisture by evaporation cannot be avoided in any case and in making calculations of water quantities allowance should be made for such losses. Experience with the soils available soon provides data that can be used to avoid duplication of operations and to estimate the excess water that must be applied to take care of evaporation losses.

The maximum thickness of soil layer that may be compacted in one operation is usually set by the specification, and on most work is 6 inches compacted depth. Some soils will not compact uniformly with certain types of rolling equipment when a loose thickness sufficient to produce 6 inches compacted depth is rolled; in such cases thinner layers must be used. The thickness for each soil type must be determined by trial and error since no test has been devised to give this information. Several small areas of soil of different thicknesses should be brought to optimum moisture content and rolled to determine the greatest thickness that may be used to compact to maximum density and the minimum number of roller trips required to produce that density.

The particular type of roller equipment used to compact embankments is of no importance if the required density is obtained and satisfactory construction progress is maintained. Sheepsfoot or tamping, smooth-faced, and rubber-tired rollers have been used with success.

Sheepsfoot or tamping rollers are used most extensively. These rollers vary in design from small single-drum rollers to the large double-drum type used on large dams and the compaction pressures range from 90 to 675 pounds per square inch. One of the chief advantages of this type of roller is that the unit load on the feet may be increased or decreased by variations in the ballast in the drum.

Tamping rollers should be of the twin-cylinder type with a frame and tongue that can be attached to a tractor in such a manner that the entire device may be either pulled or pushed in operation. The frames for the two rollers should be pivoted in a manner that will permit the rollers to adapt themselves to uneven ground surfaces and to rotate independently of one another. Cleaning teeth should be attached to the frame at the rear to prevent accumulation of soil between the tamping feet. The tamping feet should be placed in staggered rows.

Table 3 gives dimensions and weights typical of rollers in current use. This description is not intended to cover all rollers of this type in use and any roller must be judged by performance rather than by any dimensional requirements.

TABLE 3.—*Dimensions and weights typical of rollers in current use*

Item	Minimum	Maximum
Number of drums	2	2
Length of each drum (approximate) feet	4	4
Outside diameter of drum without teeth inches	38	42
Space between drums do	----	12
Length of tamper feet do	6	8
Bearing area of each foot square inches	4	13
Tamping feet per square foot of tamped area	1	2
Ground pressure under each foot pounds per square inch	100	----
Total weight pounds per inch of roller width	90	----

The tamping roller compacts the soil from the bottom of the layer toward the top and thus produces a uniform density through the entire thickness. The density of the soil layer increases up to about 10 to 12 passes of the roller for average soil conditions. If the number of passes to produce the required density exceeds 15, it indicates that the roller is too light or the layer of soil too thick and that an adjustment is necessary to produce the desired result. The compaction of clay soils usually requires the maximum weight to which the roller can be loaded. In some silty soils containing a very small amount of binder, the minimum weight of roller gives the greatest density in the least number of passes since this condition avoids the tearing of the soil by the roller feet. Tamping rollers do not operate satisfactorily in soils containing large quantities of gravel or stone particles.

In the operation of the tamping-type roller, it is important that the feet be kept free from mud and dirt. If they become clogged the efficiency of the roller is destroyed.

The smooth-faced roller compacts from the top down and usually requires from four to six passes of a 10-ton, three-wheel roller to compact a soil layer to required density in a 6-inch compacted thickness. Sandy loams having relatively low plasticity indexes can usually be compacted more economically with this type of roller than with the tamping type.

Rubber-tired rollers have not been used to any great extent on fill compaction. The information available indicates that satisfactory compaction may be obtained in sandy soils when thin layers are rolled with this type of equipment.

The compaction of embankments may be accomplished by the passage of hauling equipment, such as tractor wagons and trucks, over the soil layers during the process of construction. The distribution of equipment over the area to be compacted is difficult to control and the use of the method may result in a lack of uniformity in the density and moisture content of the soil in the finished embankment. The practice is not recommended as a substitute for rolling.

The essential factors to be given special attention in soil compaction may be summarized as follows:

1. Required moisture uniformly distributed.
2. Maximum thickness of soil layer.
3. Uniform thickness of soil layer.
4. Number of roller passes.
5. Weight of tamping rollers.
6. Cleanness of feet of tamping roller.

CONTROL TESTS SHOULD BE MADE IN THE FIELD DURING CONSTRUCTION

During processing and rolling operations, control tests should be made, the results of which will indicate the extent to which compaction has been completed. The following tests should be made in the field by the inspector during construction:

1. Compaction tests to determine moisture–density relations.
2. Moisture determinations of soil from borrow pits or cut sections.
3. Density tests of compacted soil in place.
4. Density tests of soil in place in borrow pit or cut sections.

The compaction test procedure for the determination of the moisture–density relation of soils has already been described. The data obtained by this test should be included in the soil survey report for each of the major types of soil on the project. It would be impossible, however, to anticipate at the time of the soil survey all conditions which may develop after work begins and therefore frequent compaction tests in the field laboratory are necessary in order to insure accurate control of the work. Compaction tests should be made when the soil type changes or when it may be necessary

FIGURE 18.—APPARATUS FOR DRYING SOIL BY BURNING ALCOHOL.

to use a mixture of soils to facilitate construction operations. Frequent test borings should be made with a 4-inch post auger in advance of grading operations in order to anticipate conditions and obtain samples for making compaction tests.

The moisture content of soil may be determined in the field by evaporating to dryness on a gasoline stove, by mixing the soil with alcohol and burning off the alcohol-water mixture, or by the use of the penetration needle and the moisture-penetration curve.

Evaporating to dryness may be done in accordance with the following procedure:

1. Obtain a representative sample of soil to be tested. If a metric scale is available, the sample should not be smaller than 100 grams. If an avoirdupois scale graduated by ½ ounces is used, the sample should contain at least 50 ounces.
2. Weigh sample and record weight.
3. Place sample in pan and spread to permit uniform drying. Set pan in the oven (or in a second pan) to prevent burning of soil and place on stove.
4. Dry to constant weight. The temperature of the oven should not exceed 105° C. (221° F.). Stir constantly to prevent burning.
5. After the sample has been dried to constant weight, remove from oven and allow to cool sufficiently to permit the absorption of hygroscopic moisture. Weigh dried sample and record weight.
6. Compute moisture content as follows:

$$\text{Percent moisture} = \frac{\text{weight wet soil} - \text{weight dry soil}}{\text{weight dry soil}} \times 100.$$

The alcohol burning method consists of mixing the damp soil with sufficient denatured or grain alcohol to form a slurry in a perforated metal cup, igniting the alcohol and allowing it to burn off. Three burnings of alcohol are usually required to remove all moisture from the soil. Excessive soil temperatures are not produced by this method as is evidenced by the fact that a filter paper in the perforated cup does not char. The results obtained by this method check closely with these obtained by careful laboratory drying. The apparatus is shown in figures 18 and 19.

The procedure for this method is as follows:

1. Weigh the perforated cup with the filter paper in place in the bottom. Record weight.
2. Obtain a sample which is representative of the soil to be tested. Since this method requires a sample weighing between 25 and 35 grams, a metric scale is necessary.
3. Place the sample in the perforated cup and weigh cup and sample and record weight. Weight of moist sample equals this weight minus weight of cup and filter paper.

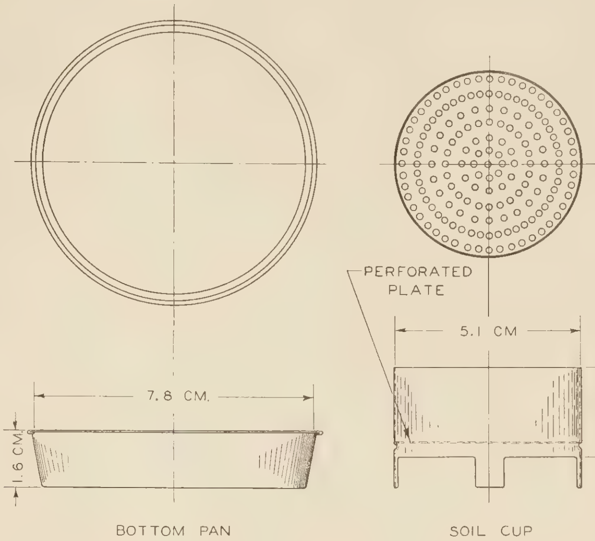


FIGURE 19.—ALCOHOL SOIL MOISTURE APPARATUS.

4. Place perforated cup in outside metal saucer and stir alcohol into the soil sample with a glass rod until a sufficient quantity has been added to produce a thin mud or slurry. Allow the stirring rod to dry and wipe soil particles clinging to it into the cup.
5. Ignite the alcohol in saucer and sample and burn off all the alcohol.
6. Repeat the process of adding alcohol and burning three times. The alcohol should be thoroughly mixed with the soil each time.
7. Weigh perforated cup and dry soil after last burning. The weight of dry sample equals this weight minus the weight of the cup and filter paper.
8. Calculate the moisture content as follows:

$$\text{Percent moisture} = \frac{\text{weight wet soil} - \text{weight dry soil}}{\text{weight of dry soil}} \times 100.$$

The apparatus shown in figures 18 and 19 may be increased in size if it seems desirable to use a larger sample. In a larger device the perforated dish should be shallow and the volume increased by increasing the diameter because a shallow sample dries faster and more uniformly and requires less alcohol.

MOISTURE DETERMINATION WITH PENETROMETER DESCRIBED

An approximation of the moisture content of soil for which moisture-penetration curves are available may be made with the standard soil penetrometer by the following procedure:

1. Place compaction mold on firm foundation.
2. Compact two layers of soil in the mold in accordance with the standard procedure used in the compaction test.
3. Record the pressure required to force the penetrometer needle into the compacted soil. The readings for three trials should be recorded and averaged. Convert the readings to pounds per square inch.
4. Read moisture content corresponding to unit pressure from the moisture-penetration curve for the sample being tested.

The evaporation to dryness method and the alcohol burning method of obtaining the moisture content of soil in the field have given satisfactory results. The first method is somewhat cumbersome and requires constant attention to prevent burning of the sample. Since large samples can be dried by this method, inaccuracies due to sampling may be reduced to a minimum. It is also adapted for use with materials containing large aggregates such as sand-gravel mixtures.

The alcohol method cannot be used for coarse granular mixtures unless the size of the containers is increased accordingly. The use of a large container will require the use of a large quantity of alcohol which would make the cost of the test prohibitive. The quantity of alcohol required for each burning is approximately twice the volume of the moisture in the sample. For example, a 100-gram sample containing 20 percent moisture would require 40 cubic centimeters of alcohol for each burning or a total of 120 cubic centimeters for complete drying.

The alcohol method has the advantages of being easy to use and utilizing equipment that does not easily get out of repair and which is compact, and low in cost. Several of the devices can be operated simultaneously without danger of burning the soil.

The penetrometer method of moisture determination can be used only in fine-grain soils and gives approximate values. The method is useful as a control test because the approximate moisture contents can be checked rapidly. The method is not used to replace the drying tests but may be considered as supplemental to them.

The determination of the density of compacted soil and of the undisturbed soil in excavation areas as the construction of an embankment proceeds is important as a control measure, as a means of checking the work against specification requirements, and for the calculation of the shrinkage factors used in estimating the volume of excavation necessary to produce embankments of given dimensions.

Density tests of soil in place may be made by measuring the weight, volume, and moisture content of undisturbed samples or by measuring the volume occupied by a disturbed sample and recording the weight and moisture content of the soil removed from that volume.

Undisturbed samples may be cut with hand tools and tested by the following procedure:

1. A sample is obtained by marking an area of the same size as the sample desired and digging the soil from around it with some sharp tool such as a knife, spatula or small trowel. A spade may be used if care is exercised not to disturb the core. The sample should be 4 to 5 inches in diameter and the full depth of the lift.
2. Immediately upon removal of the core a representative sample should be removed for moisture determination. The size of the moisture sample will depend upon the method to be used in the field laboratory for drying the moisture samples.
3. Trim loose material from soil core, weigh, and record weight to nearest ½ ounce.
4. Determine moisture content by drying moisture sample.
5. Immerse sample in hot paraffin until coated, remove, cool, and weigh. The gain in weight represents the weight of paraffin and the volume of the coating is calculated using 55 pounds per cubic foot as the weight of paraffin.
6. Weigh coated sample in water, record weight and calculate volume or measure volume of water displaced by means of a suitable overflow device. Deduct the volume of the paraffin coating.
7. Compute wet and dry density by the following formulas:

$$\text{Wet weight per cubic foot}=\frac{\text{weight of wet sample}}{\text{volume of sample}}.$$

$$\text{Dry weight per cubic foot}=\frac{\text{wet weight per cubic foot}}{1+\frac{\text{percent moisture}}{100}}.$$

For example assume wet weight of soil sample = 8 pounds; volume of sample = 0.06 cubic foot; and moisture content = 15 percent; wet weight per cubic foot $=\frac{8}{0.06}=133$ pounds; and dry weight per cubic foot $=\frac{133}{1+0.15}=115.7$ pounds.

Undisturbed samples may also be obtained by driving a tube sampler into the soil layer. If the volume of the sampler is known, the determination of the volume of the sample becomes unnecessary. Care must be exercised in the use of the method to avoid disturbance of the soil.

METHODS OF DETERMINING DENSITY OF SOIL LAYER GIVEN

The density of a soil layer may be determined by finding the weight of a disturbed sample and measuring the volume occupied by the sample prior to removal. This volume may be measured by filling the space with a weighed quantity of a medium of predetermined weight per unit volume. Sand, heavy lubricating oil or water in a thin rubber sack may be used as a medium for measuring the volume formerly occupied by the sample. Except for the determination of the weight per cubic foot of the medium, the three procedures are the same and therefore the one using sand will be described in detail. It is as follows:

1. Determine weight per cubic foot of the dry sand to be used by filling a measure of known volume. The height and diameter of the measure used should be approximately equal and its volume should be not less than $\frac{1}{10}$ cubic foot. The sand should be deposited in the measure by pouring through a funnel or from a measure with a funnel spout from a fixed height. The measure is filled until the sand overflows and the excess is struck off with a straightedge. The weight of the sand in the measure is determined and the weight per cubic foot computed and recorded.
2. Remove all loose soil from an area large enough to place a box similar to the one shown in figure 20 and cut a plane surface for bedding the box firmly.
3. With a soil auger or other cutting tools bore a hole the full depth of the compacted lift.
4. Place in pans all soil removed, including any spillage caught in the box. Remove all loose particles from the hole with a small can. Extreme care should be taken not to lose any soil.
5. Weigh all soil taken from the hole and record weight.
6. Mix sample thoroughly and take sample for moisture determination.
7. Weigh a volume of sand in excess of that required to fill the test hole and record weight.
8. Deposit sand in test hole by means of a funnel or from a measure by exactly the same procedure as was used in determination of unit weight of sand until the hole is filled almost flush with original ground surface. Bring the sand to the ground level by adding the last increments with a small can or trowel and testing with a straightedge.

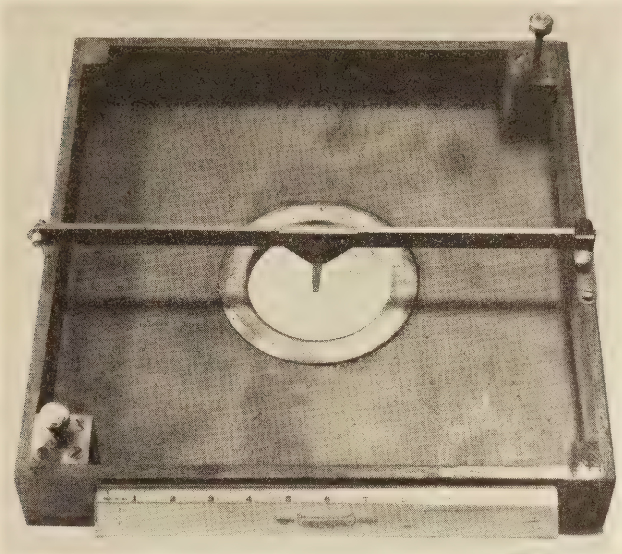


Figure 20.—Soil Tray for Use with Post Auger in Soil Density Determinations.

9. Weigh remaining sand and record weight.
10. Determine moisture content of soil samples.
11. Compute dry density from the following formulas:

$$\text{Volume of soil} = \frac{\text{weight of sand required to replace soil}}{\text{weight per cubic foot of sand}}.$$

$$\text{Wet weight per cubic foot} = \frac{\text{weight of soil}}{\text{volume of soil}}.$$

$$\text{Dry weight per cubic foot} = \frac{\text{wet weight per cubic foot}}{1+\frac{\text{percent moisture}}{100}}.$$

For example assume weight per cubic foot of sand=100 pounds; weight of wet soil from auger hole=5.7 pounds; moisture content of soil=15 percent; and weight of sand to fill auger hole=4.5 pounds. Then volume of soil from hole$=\frac{4.5}{100}=0.045$ cubic foot; weight per cubic foot of wet soil$=\frac{5.7}{0.045}=126.7$ pounds; and weight per cubic foot of dry soil$=\frac{126.7}{1+\frac{15}{100}}=110$ pounds.

Assume that optimum moisture for this soil equals 15 percent and maximum density equals 115 pounds per cubic foot, then the compaction in the layer tested is $\frac{110}{115}=95.7$ percent.

If the specifications require not less than 95 percent of maximum density at optimum moisture, the compaction is satisfactory but very close to minimum requirements.

When the sand funnel device shown in figure 21 is used to determine the volume of the soil removed from the test hole, the volume of the jar above the valve may be determined by filling the apparatus with water, closing the valve, pouring off water retained in the large funnel, and weighing. The volume may be computed by dividing the weight of water in the jar by weight per cubic foot of water (62.4 pounds). After the volume of the apparatus is known, the weight of sand required to fill it may be determined and the unit weight computed. The device is used by placing the funnel over the hole, opening the valve and allowing the sand to flow until it stops. The valve is closed and the weight of sand left in the jar is determined. This value subtracted from the total weight of sand in the device gives the weight required to fill the hole and the cone. The weight of sand in the cone can be found by weighing the apparatus, placing it on a flat surface, opening the valve, allowing the sand to flow until it stops and closing the valve. The weight of sand in the cone equals the difference in weight of the apparatus before and after the filling operation.

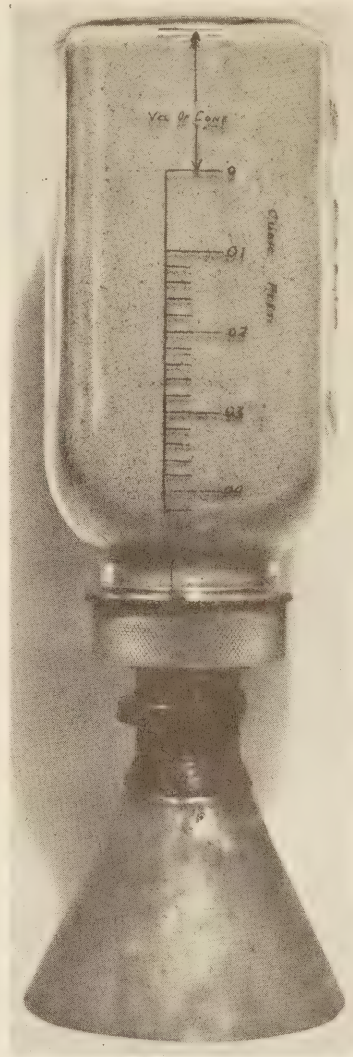


Figure 21.—Sand Jar with Funnel for Use in Soil Density Determinations.

The jar may be calibrated to show cubic feet of sand removed as shown in figure 21 so that weighing is not necessary in the determination of soil volume. Such calibration should be made very carefully and requires more equipment than is usually available in a field

Figure 22.—Oil Jar and Pump for Use in Soil Density Determinations.

Figure 23.—Rubber Sack with Measuring Jar for Use in Soil Density Determinations.

laboratory. When volumetric measurements are used, the readings must be made carefully and care must be exercised not to compact the sand during the operation.

Any clean sand having rounded particles all of one size (usually passing the No. 20 and retained on the No. 30 sieve) may be used in this test. Standard Ottawa sand is used to a large extent but is not required. The sand may be salvaged after each test but should be rescreened before being used again. The use of slightly damp sand should be avoided because of the error introduced by bulking.

Heavy lubricating oil (S. A. E. 30 or 40) may be used instead of sand in the above test. The procedure and method of computing the results are the same. The weight per cubic foot of the oil may be found by weighing a measured quantity or by computing it from the specific gravity if that constant is known. The oil is removed from the hole with a suction pump and may be used until it becomes contaminated with soil particles to the extent that the weight may be changed. A calibrated container may be used if means are available for accurate calibration and etching of the quantities on the glass. The use of a device calibrated as shown in figure 22 is convenient due to elimination of weighing procedures and is accurate when the readings are carefully made. The suction pump shown in figure 22 is the type ordinarily used in the recovery of the oil.

The apparatus shown in figure 23 consists of a rubber pouch attached to a calibrated glass container and may be used to measure the volume of the space from which a disturbed sample is taken. The device comprises a closed system and is very convenient due to the fact that the necessity for the handling of oil or sand is eliminated. The volume of the rubber pouch must be determined accurately and correction for its volume made in the readings taken. To insure the filling of the entire volume from which the soil sample was taken, air pressure is introduced into the jar by means of the small bicycle pump shown in figure 23. The pressure is easily determined by trial since the water level will not be lowered by slight increases in pressure after the rubber has expanded into the irregularities of the hole.

The use of the sand funnel device of figure 21 or the calibrated container for measuring the volume of oil, figure 22, are limited to fine-grained soils where irregularities in volume due to large aggregate particles do not occur. The sand funnel cannot be placed over a hole irregular in shape and the quantity of the oil in the calibrated container is usually too small to fill the excess volume caused by the removal of stones, etc. It is obvious that the rubber pouch device can be used only in fine-grained soils since the rubber cannot be expanded into a test hole of irregular shape.

SOIL MASS IN EMBANKMENTS CONSISTS OF SOIL PARTICLES AND AIR AND WATER VOIDS

The form shown in figure 24 is suggested for use in recording field data obtained in the inspection of the compaction of embankments.

For the correct interpretation of soil data, the relationship of the soil particles, water, and air voids in the soil mass must be understood. The following fundamental facts may be used to interpret the test data correctly.

A soil mass as it exists in an embankment is made up of soil particles and voids. Part of the void space contains air and part of it contains water.

Let V_s=volume of soil particles in a unit volume of soil;

V_v=total volume of air and water;

V_w=volume of the voids filled with water;

V_a=volume of the voids filled with air;

then $V_s+V_v=V_s+V_w+V_a=$unity;

and let G=specific gravity of soil particles;

w=percent moisture by dry weight of soil;

W=wet weight per cubic foot of soil;

W_0=dry weight per cubic foot of soil;

a=percent moisture by dry weight of soil to fill all the voids (V_v);

and assume that W=124 pounds per cubic foot;

w=17 percent;

G=2.70;

Project ______________________ Date ____________

Location ______________________ Operators ____________

Field test No.						
Location of tests : Station						
: Reference to C L						
: Elevation						
Elevation : Original ground						
: Finished grade						
Type of roller						
No. of passes with roller						
Density determination						
A. Weight per cu. ft. of sand (or oil) =	100					
B. Weight sand (or oil) + weight of container =	18					
C. Weight sand (or oil) left in container + container =	13.5					
D. Weight sand (or oil) in auger hole (B - C) =	4.5					
E. Volume of auger hole (D ÷ A) =	.045					
Weight of wet soil from auger hole + weight of container =						
Weight of container =						
F. Weight of wet soil from auger hole =	5.7					
G. Weight per cubic foot of wet soil in fill (F ÷ E) =	126.7					
H. Weight of dry soil in 1 cubic foot of fill =	110					
K. Maximum dry weight per cu. ft. from Lab. No. =	115					
L. Compaction = (H ÷ K) x 100 =	96					
Moisture determination						
Dish and damp soil						
Dish and dried soil						
Weight moisture						
Weight dish (No.)						
Weight dry soil						
M. Percentage moisture	15					

FIGURE 24.—FORM OF REPORT ON EMBANKMENT COMPACTION.

then $W_0=\dfrac{W}{1+\dfrac{w}{100}}=\dfrac{124}{1+\dfrac{17}{100}}=106$ pounds per cubic foot.

$V_s=\dfrac{W_0}{G\times 62.4}=\dfrac{106}{2.70\times 62.4}=$ 0.629 cubic foot of solid particles in each cubic foot of soil.

$V_v=1-V_s=1-0.629=$ 0.371 cubic foot of combined air and water voids in each cubic foot of soil.

Since the percentage of moisture is known, the volume of the water in the voids may be calculated thus:

$V_w=\dfrac{\dfrac{w}{100}\times W_0}{62.4}=\dfrac{\dfrac{17}{100}\times 106}{62.4}=0.288$ cubic foot of water in each cubic foot of soil.

$V_a=V_v-V_w=0.371-0.288=0.083$ cubic foot of air n each cubic foot of soil.

Percentage of air voids by volume $=V_a\times 100=8.3$.

The volume of air voids may also be calculated from the following:

$$V_a=1-V_s-V_w$$

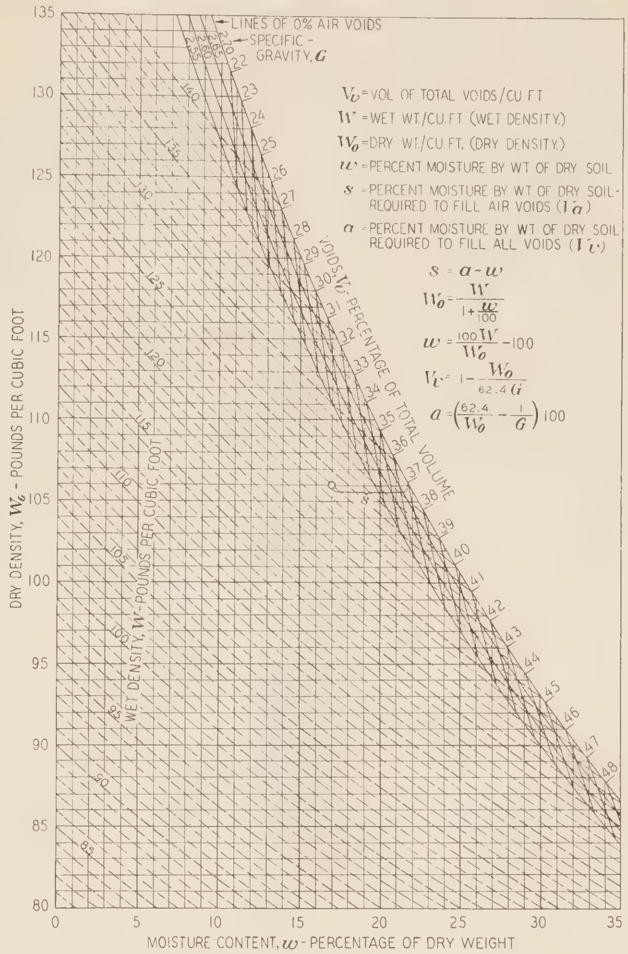
$$=1-\frac{W_0}{62.4G}-\frac{wW_0}{100\times 62.4}=1-\frac{W_0}{62.4}\left(\frac{1}{G}+\frac{w}{100}\right)$$

When the air voids are zero ($V_a=0$), the soil is saturated and $V_w=V_v=0.371$ cubic foot of water in each cubic foot of soil.

Percent moisture by volume for zero air voids $=V_v\times 100=37.1$.

The moisture content by volume for zero air voids may be converted to a weight basis by means of the following equation:

$$a=\left(\frac{62.4}{W_0}-\frac{1}{G}\right)100$$



EXAMPLE :- GIVEN w=17, W=124, G=2.70 (POINT o). THEN, W_0 = 106; V_v=0.371, a =21.9, s = 4.9

FIGURE 25.—CHART OF SOLIDS-WATER-VOIDS RELATIONS OF SOIL MASSES.

For the example above

$$a=\left(\frac{62.4}{106}-\frac{1}{2.70}\right)100=21.9$$

The relationship between the dry weight per cubic foot of soil and the percentage of moisture by weight necessary to fill the voids is useful in checking the values obtained by testing the density of the soil in place. Since the soil in place always contains some air voids, the percentage of moisture by weight of the soil cannot exceed the moisture content required to reduce the air voids to zero. Also, if the computed weight per cubic foot of soil in the embankment is higher than the weight when the air voids are zero, it is obvious that an error has been made in the determination of the weight or the moisture content. The test results can be checked conveniently by the use of curves constructed by plotting the moisture contents by weight for the zero air voids conditions against the dry weights per cubic foot for several specific gravities and drawing a smooth curve through the points. The dry weight per cubic foot and moisture content of soil can be plotted on such a chart with a minimum of effort and errors in testing procedure located and corrected without loss of time.

A series of curves for the more common specific gravities is shown in figure 25. These curves are also suitable for use in calculating the dry weight per cubic foot of soil from results of tests to determine the wet weight per cubic foot and the moisture contents of the soil in an embankment. As a check on test results, when the moisture content of the soil is plotted against the dry weight per cubic foot, the point should fall to the left of the zero air voids curve. If it does not, the test data are obviously in error.

RELATION OF EXCAVATION AND EMBANKMENT DENSITIES OFTEN USEFUL

The balance factor in earthwork is the ratio of the density of the embankment to the density of the cut or excavation. It involves a study of densities in the cut section as well as in the fill section. Accurate knowledge of cut densities is sometimes quite useful to the engineer in determination of the quantity of excavation in instances where borrow pits have been flooded and silted in after excavation and in instances where pits have been badly eroded or washed out. They are often useful in calculation of hydraulic excavation. The accuracy of earth quantities as measured by the method of average end areas obtained by cross sections is sometimes questioned. When the volumes calculated from cross sections are in doubt, data on both cut and fill densities are of considerable value in checking the final quantities.

Earthwork quantities are directly related to densities. That is, the cubic yards of embankment which are obtained from a given number of cubic yards of excavation are directly related to the density of the embankment and that of the excavation.

The formula for determining the balance factor may be derived as follows:

Let A=volume of excavation;

B=volume of embankment;

W=weight of material necessary to produce a given volume of excavation or embankment;

D_f=dry density in pounds per cubic foot of embankment;

D_c=dry density in pounds per cubic foot of excavation; and

$\frac{D_f}{D_c}$=balance factor;

then, since density$=\frac{\text{weight}}{\text{volume}}$,

$$\frac{W}{B}=D_f \tag{1}$$

$$\frac{W}{A}=D_c \tag{2}$$

$$W=BD_f \tag{3}$$

$$W=AD_c \tag{4}$$

$$AD_c=BD_f \tag{5}$$

then, balance factor:

$$\frac{D_f}{D_c}=\frac{A}{B} \tag{6}$$

Assume that the cubic yards of excavation necessary to produce an embankment of 5,000 cubic yards is to be calculated.

Then A=unknown.

B=5,000 cubic yards.

Assume $D_f=106$

$D_c=97$

Then substituting in equation 6,

$$A=5000\times\frac{106}{97}$$

$$A=5{,}464$$

$$\frac{D_f}{D_c}=\frac{106}{97}=1.093 \text{ (balance factor).}$$

The earth shrinkage from excavation to embankment is equal to the amount, in percent, that the volume of excavation exceeds the volume of embankment. It is calculated from the equation,

$$S=\frac{(D_f-D_c)}{D_c}100$$

where S=shrinkage, in percent

and

$$S=\frac{106-97}{97}100=9.3 \text{ percent.}$$

In the course of ordinary construction, when ordinary soils are taken from shallow excavation (borrow pits and shallow cuts) the balance factor will, if good compaction is being obtained, be greater than one (1.000). In some instances shales have been encountered where it has been either impossible or impractical to consolidate the material in the embankment to the very dense state in which the shale occurs in its natural bed or layer. Under such conditions a balance factor of less than 1 does not necessarily signify poor compaction.

Similarly, when soils are taken from very deep cut sections where they exist in a very compact condition, it has been found that even under good rolling procedure the resulting embankment density is lower than the density of the soil in the excavation.

Nevertheless, when either of the above conditions exists it should be thoroughly investigated to determine whether or not the best compaction is being obtained.

In using the balance factor to determine quantities of excavation, it should be kept in mind that factors such as wastage in hauling, loss of material by blading off grass and weeds, loss due to erosion by floods and any other losses or gains should be taken into consideration. With these factors in mind, it is easier to account for the discrepancies which might exist between the final results.

The conditions on earth work projects vary so widely that it is difficult to set forth the number of tests that will be necessary for adequate control of the compaction of embankments. Common practice requires that when the soil and moisture conditions are uniform, a minimum of four density and moisture tests should be made in each 8-hour day but not less than one test should be made for each 500 cubic yards of excavation. When soil and moisture conditions vary, the number of tests will have to be increased sufficiently to insure accurate control. Actually, the number of tests required will have to be determined by experience. In starting a project frequent tests should be made to establish in the mind of the inspector the appearance and consistency of the various soils when they are in most suitable condition for compaction. The inspector on important earth work must take his job seriously and learn by frequent testing the best methods to use to produce a good embankment from the available material.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JANUARY 31, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$6,437,162	$3,199,140	227.6	$3,958,624	$1,964,205	113.2	$488,210	$242,350	2.9	$2,760,201
Arizona	1,393,527	979,140	58.6	1,444,743	900,035	50.0	384,302	254,599	8.4	2,160,619
Arkansas	3,460,021	1,590,151	56.6	1,173,174	585,470	60.1	66,680	33,340	.1	1,863,552
California	7,568,885	4,040,283	126.0	6,165,635	3,536,378	73.0	687,431	394,653	20.8	4,945,511
Colorado	2,310,848	1,303,814	154.6	3,200,996	1,840,260	150.4	1,120,230	626,362	40.6	2,616,120
Connecticut	1,419,554	696,181	17.1	1,715,156	821,787	18.4	481,485	239,471	9.4	1,228,870
Delaware	365,525	177,441	9.5	659,559	326,584	16.6	268,040	134,020	8.4	1,509,078
Florida	1,167,179	582,930	65.3	2,281,618	1,144,037	28.4	1,353,864	843,856	17.9	3,200,526
Georgia	2,772,525	1,384,970	100.7	6,789,779	3,404,640	265.9	3,649,716	1,824,858	148.9	7,080,615
Idaho	1,782,936	1,086,595	93.7	1,301,603	803,077	63.1	58,224	36,000	.1	2,189,363
Illinois	3,876,581	1,917,141	87.2	7,152,266	3,575,561	124.1	1,404,200	702,100	6.2	7,152,928
Indiana	4,525,625	2,095,222	74.4	5,305,469	2,573,969	75.6	2,191,200	1,095,600	32.7	2,868,489
Iowa	3,627,220	1,735,171	151.9	4,366,352	1,906,282	138.4	800,741	97,650	23.6	2,531,791
Kansas	4,526,360	2,289,183	250.2	4,966,257	2,491,064	237.6	2,158,829	783,867	76.7	5,281,813
Kentucky	3,771,487	1,861,214	139.1	6,386,223	3,051,471	125.1	2,083,762	974,967	17.2	2,005,284
Louisiana	1,013,101	506,526	23.9	1,835,052	909,108	38.1	2,553,917	1,251,767	56.3	4,551,560
Maine	954,535	472,598	26.8	2,070,392	1,061,696	27.8	78,610	39,305	.1	1,122,228
Maryland	2,737,267	1,367,529	29.6	3,474,909	1,480,154	15.0	35,000	17,500		1,587,207
Massachusetts	2,348,774	1,177,018	17.3	2,223,103	1,142,862	14.9	1,173,468	583,147	8.4	3,911,401
Michigan	8,312,252	4,065,652	173.7	2,855,748	1,427,874	44.9	1,831,600	907,500	18.5	3,517,357
Minnesota	4,480,915	2,219,233	378.0	9,536,083	4,724,469	388.4	368,018	184,009	20.1	3,470,206
Mississippi	3,584,667	1,784,718	208.8	5,310,224	2,603,712	286.5	197,500	100,000	6.1	2,306,675
Missouri	5,045,503	2,492,307	160.1	10,036,428	5,134,979	190.0	2,989,937	1,105,698	36.5	4,643,464
Montana	2,181,671	1,233,971	117.1	3,728,153	2,181,315	160.3	699,027	401,686	68.8	4,470,051
Nebraska	2,210,975	1,085,636	237.7	5,909,661	2,971,285	519.7	709,046	354,523	37.8	4,184,920
Nevada	2,244,854	1,946,128	110.6	742,339	643,031	22.0	274,686	238,701	3.4	1,385,808
New Hampshire	339,179	177,926	6.0	1,245,854	596,796	14.8	447,973	335,980	5.4	934,962
New Jersey	3,009,259	1,475,339	26.6	2,964,992	1,482,416	16.2	23,910	11,955		3,030,650
New Mexico	1,560,262	960,958	102.0	1,152,059	744,921	74.1				2,916,946
New York	9,597,967	4,717,100	126.7	7,910,755	4,906,145	80.6	1,083,700	633,400	12.8	5,430,481
North Carolina	3,292,024	1,629,777	136.6	3,454,449	1,730,110	145.6	1,138,663	573,423	27.0	3,716,426
North Dakota	3,287,136	1,802,745	287.6	2,572,786	1,323,524	203.7	2,421,360	1,214,050	205.5	4,280,623
Ohio	9,533,041	4,760,070	91.6	10,664,162	5,090,986	69.3	5,797,060	2,247,378	37.2	4,690,951
Oklahoma	2,097,080	1,054,494	107.1	2,545,022	1,342,949	41.4	2,106,280	1,102,854	77.1	6,267,587
Oregon	2,674,796	1,585,246	72.9	3,192,624	1,692,738	69.7	259,274	113,350	7.5	1,826,243
Pennsylvania	9,915,126	4,916,577	98.5	8,804,300	4,281,780	65.5	2,957,859	1,475,491	24.9	4,918,975
Rhode Island	1,196,941	596,510	10.0	644,776	322,368	4.8	644,448	322,224	2.0	940,591
South Carolina	1,946,604	891,797	87.2	3,971,331	1,823,981	90.0	994,153	379,881	26.8	2,352,037
South Dakota	1,985,584	1,149,084	232.2	5,054,628	3,248,383	578.7	644,800	381,560	69.6	2,941,136
Tennessee	3,242,183	1,617,780	94.7	4,851,864	2,572,126	84.0	1,199,978	647,546	34.4	3,954,218
Texas	8,467,428	4,145,126	447.9	13,733,944	6,559,286	464.1	2,669,301	1,120,520	88.4	9,492,388
Utah	1,206,046	907,446	53.8	1,761,015	1,325,602	41.3	72,348	54,000	5.4	1,298,207
Vermont	823,135	408,394	28.8	1,205,174	595,537	20.7	36,906	18,453	.3	552,510
Virginia	2,999,775	1,470,809	70.6	4,387,238	2,048,126	61.4	35,490	17,745	.4	2,937,659
Washington	1,474,775	765,618	20.2	2,809,895	1,503,654	36.9	43,686	23,400	1.0	2,244,808
West Virginia	2,778,324	1,380,071	51.4	2,344,962	1,162,723	28.1	463,776	229,588	3.1	2,205,205
Wisconsin	2,201,240	1,080,730	92.7	5,546,496	2,597,643	163.9	1,262,339	454,400	44.7	4,831,516
Wyoming	1,426,636	921,287	148.3	1,793,023	1,198,146	123.5	34,714	22,321	.1	1,475,803
District of Columbia	594,036	293,515	3.3	721,662	396,682	1.1	600,000	382,500	.2	306,857
Hawaii	133,296	66,648	2.8	1,065,560	700,306	10.9	180,505	161,032	3.5	1,864,746
Puerto Rico	417,697	206,815	3.6	1,875,051	924,890	16.6	342,531	170,280	1.7	850,861
TOTALS	160,319,519	82,271,754	5,499.2	200,863,168	103,377,123	5,754.4	53,568,777	25,560,860	1,348.9	160,808,023

Note: Includes apportionments for fiscal year 1943.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF JANUARY 31, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	NUMBER: Grade Crossing Structures Reconstructed	NUMBER: Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	NUMBER: Grade Crossing Structures Reconstructed	NUMBER: Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	NUMBER: Grade Crossing Structures Reconstructed	NUMBER: Grade Crossings Protected by Signals or Otherwise	
Alabama	$151,956	$151,436	2	4	2	$383,125	$381,103	5	2	2	$52,335	$52,335	2		5	$1,129,771
Arizona	168,283	168,266	1			141,369	132,678		2		13,255	13,255			2	234,281
Arkansas	452,984	451,679	5		1	172,925	171,215	1		2	28,621	28,621			10	660,187
California	830,690	641,118	2	1	3	870,516	868,399	8			15,678	15,678			5	2,319,005
Colorado	5,685	5,646				664,333	664,333	7			21,042	21,042			10	730,378
Connecticut	166,222	165,415	2			61,712	60,676		1		231,374	222,740		1		525,593
Delaware	91,204	91,204	1		1	191,599	189,867	1	1		508,406	321,785	2			175,142
Florida	108,679	108,679			18	727,448	725,268	8		4	205,901	205,840			18	875,288
Georgia	547,990	547,990	7	1	3	939,400	939,400	5	7	10	959,078	959,078	3	4	16	1,515,472
Idaho	25,827	21,580			2	322,273	313,602	4	1		6,212	6,212			3	427,671
Illinois	685,009	563,091	2		80	1,661,232	1,566,997	8	1	9	426,384	407,434	1	1	37	2,557,257
Indiana	600,254	587,767	6		18	466,062	460,353	2	1	20	100,783	100,783			26	1,166,559
Iowa	338,129	324,372	3	1	24	1,459,107	1,206,930	10	2	17	180,636	179,225			45	661,098
Kansas	63,041	62,622	2		5	677,142	677,142	8		2	216,248	172,678	4		10	1,358,519
Kentucky	1,040,740	1,038,377	8		1	512,092	512,092	5	1							407,268
Louisiana	6,965	6,965				586,220	586,220	8			481,835	480,667	4		4	923,735
Maine						363,086	363,086	2			8,680	8,680			3	288,623
Maryland	488,250	456,457	2	2	10	868,458	724,660	4		4	48,775	48,775			8	399,876
Massachusetts	346,270	335,829	1		1	774,431	773,559	5	2		763,830	763,830	2	1		1,267,765
Michigan	1,300,420	1,293,532	3	4	13	262,616	262,616	1		9	338,780	308,619	1		13	1,324,681
Minnesota	532,379	532,063	5	3	1	1,057,043	1,057,043	5	5		76,944	76,944		1	8	1,194,810
Mississippi	209,275	209,275	2		1	887,709	887,709	10	1	9	25,808	25,808			2	644,407
Missouri	120,702	120,702		2		1,922,921	1,467,501	6	2		464,353	464,353	2	2		1,489,049
Montana	141,549	141,549	2		2	99,778	99,778	1								733,051
Nebraska	181,040	180,663	1	1	20	1,164,361	1,164,361	22		3	13,020	13,020			3	449,496
Nevada	119,580	119,580	2			56,484	56,484	2			30,725	30,725			10	192,599
New Hampshire	207,015	199,138	4		2	162,506	162,181	2	2	3						313,808
New Jersey	845,837	844,536	4	1	1	629,879	504,329	3	1	1	354,985	295,560	1	2		925,870
New Mexico						68,342	68,342	1			259,103	252,069	3		1	521,753
New York	2,200,333	2,169,097	2	12		2,184,596	2,139,457	3	9		502,645	464,285		1		3,700,290
North Carolina	495,485	495,485	2	4	23	203,171	200,293		3	1	237,433	237,433	1	1	21	1,307,949
North Dakota	174,472	173,937	4			600,080	587,143	6			223,120	223,120	2			640,117
Ohio	1,541,619	1,526,404	8	1	5	2,770,192	2,446,470	10		3	401,060	196,930	1		5	1,701,412
Oklahoma	174,980	171,185	1		28	854,619	851,209	6		6	364,715	326,673	3		5	1,559,790
Oregon	419,536	355,255	4		3	203,552	187,715	2			2,733	2,733			2	442,391
Pennsylvania	1,275,304	1,274,976	11	1		3,632,907	3,587,997	17			359,074	359,074	2			2,514,389
Rhode Island	205,241	205,241		1		3,655	3,655									273,921
South Carolina	342,530	329,197	6		20	193,969	191,669	1	3		300,375	166,701	2	2	11	972,879
South Dakota	507,443	507,443	13		9	517,942	501,992	9			41,200	41,200	2			831,331
Tennessee	301,580	289,686	3	3		1,107,220	1,107,220	6			168,376	168,376	1	1	2	1,157,544
Texas	1,121,706	1,107,973	14			1,481,277	1,469,943	14		4	87,310	87,050	1		1	2,371,290
Utah	52,721	51,979	2		17	72,872	72,872	1		10	62,710	62,710			24	359,438
Vermont	18,683	18,671			4	322,869	293,090	2								103,978
Virginia	96,542	96,542		2	1	778,475	758,515	6	3	1	38,291	38,291	1		3	952,660
Washington	170,788	170,788	1	2	1	1,258,979	562,305	7		1	7,919	7,919			3	436,261
West Virginia	253,143	247,512	3		6	654,510	654,510	6	1	2	3,330	3,330			1	776,753
Wisconsin	467,875	438,879	2		36	574,972	573,984	3	2	9	15,484	15,484			3	1,679,851
Wyoming	483,177	468,524	5		1	1,974	1,974				8,416	8,416			6	428,019
District of Columbia	2,193	2,193				299,675	275,206		1							103,351
Hawaii	187,619	187,618	2			214,170	213,655	2								282,757
Puerto Rico	103,629	102,980	1			639,340	632,516	9			141,279	140,190	2			363,014
TOTALS	20,372,574	19,761,096	151	46	363	36,725,185	34,361,314	254	54	132	8,798,261	8,025,671	43	17	326	48,373,297

Note: Includes apportionments for fiscal year 1943.

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1932. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1939. 10 cents.

Work of the Public Roads Administration, 1940.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

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No. 191MP. . Roadside Improvement. 10 cents.

No. 272MP. . Construction of Private Driveways. 10 cents.

No. 279MP. . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

Highways of History. 25 cents.

Specifications for Construction of Roads and Bridges in National Forests and National Parks. 1 dollar.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP. . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

Indexes to PUBLIC ROADS, volumes 6–8 and 10–21, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF JANUARY 31, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$1,390,472	$692,608	62.5	$520,331	$284,000	25.2	$212,180	$100,080	6.0	$580,971
Arizona	125,776	91,405	13.2	137,116	101,803	.8	126,598	61,439	6.8	543,761
Arkansas	610,518	233,221	33.1	348,210	174,039	26.3	135,644	67,822	.2	339,836
California	760,788	438,134	17.8	975,209	724,693	8.1				1,077,518
Colorado	150,002	84,134	20.7	129,755	72,649	2.1	152,387	35,323	5.0	530,458
Connecticut	298,035	136,331	6.1	266,247	115,938	4.8				199,604
Delaware	81,076	38,116		222,731	110,890	12.3	102,873	37,617	3.9	247,133
Florida	498,886	249,443	4.7	666,633	338,767	7.1	191,500	95,662	5.9	387,671
Georgia	455,428	212,714	36.9	1,196,017	690,358	74.4	496,996	248,498	50.8	1,126,523
Idaho	285,649	173,225	26.2	177,185	108,985	7.7	78,125	48,303	5.9	308,602
Illinois	1,073,033	518,949	59.4	1,084,660	542,330	47.5	152,500	76,250	17.7	906,871
Indiana	611,250	305,625	39.8	1,128,455	531,071	46.7	189,600	94,800	6.4	926,752
Iowa	586,080	276,641	144.8	409,422	175,208	68.3	346,551	161,825	60.5	587,786
Kansas	550,267	276,484	84.6	1,834,382	919,358	111.6	499,225	249,613	37.7	1,042,437
Kentucky	1,161,015	321,329	83.2	1,063,884	274,908	32.1	344,462	94,400	14.1	397,091
Louisiana	564,708	230,289	20.6	7,700	3,850		289,362	138,761	21.5	702,038
Maine	77,540	38,770	3.6	235,218	117,609	10.6	16,850	2,714	.4	161,104
Maryland	473,000	236,500	19.5	333,376	166,513	3.1				345,564
Massachusetts	179,789	93,569	4.1	663,233	352,683	10.1				572,304
Michigan	1,129,971	556,411	71.4	748,998	374,499	26.7	435,870	217,935	14.2	641,630
Minnesota	1,595,353	800,819	217.1	973,916	484,197	86.9	300,776	149,988	30.1	603,968
Mississippi	712,594	356,297	33.2	1,316,867	592,299	60.9	242,200	85,000	11.5	488,205
Missouri	394,608	195,649	49.7	889,132	426,978	89.7	253,830	96,986	39.9	1,052,689
Montana	377,420	214,407	58.5	292,724	170,722	31.0	13,569	7,715	4.5	889,999
Nebraska	352,367	176,722	42.4	493,205	251,216	71.7	31,940	15,970	9.2	684,035
Nevada	225,871	191,918	24.1	92,413	60,816	4.6	99,973	79,290	4.5	229,165
New Hampshire	152,914	75,436	4.7	241,629	119,915	3.6				152,254
New Jersey	446,840	219,205	7.2	466,582	257,385	14.5	51,500	25,750	1.8	568,852
New Mexico	408,981	255,920	42.6	346,212	223,860	20.2				296,853
New York	957,018	470,060	28.8	894,946	488,111	14.3	485,666	242,833	1.5	1,095,866
North Carolina	333,240	168,620	34.9	536,151	294,407	36.3	69,820	20,000	5.0	680,994
North Dakota	29,802	15,664	2.4	3,434	3,434		808,050	793,860	42.7	759,120
Ohio	1,698,902	848,314	56.4	855,410	475,675	11.0	177,160	88,580	6.6	1,387,630
Oklahoma	363,069	191,497	21.6	64,572	34,093	9.9	903,706	477,157	64.1	1,083,038
Oregon	463,659	243,290	41.8	459,611	217,324	28.5	30,482	18,000	1.3	405,495
Pennsylvania	1,673,012	835,832	31.7	834,647	406,931	16.3	73,588	36,794	1.8	777,528
Rhode Island	220,879	111,427	2.6	14,694	10,697					138,411
South Carolina	787,356	307,866	54.6	221,700	79,945	.5				397,720
South Dakota	32,130	18,006	15.2	3,622	3,622		1,143,430	1,047,600	114.5	766,698
Tennessee	333,033	164,824	10.8	1,430,720	715,360	48.5	200,802	100,401	5.3	770,082
Texas	1,054,286	512,692	109.2	978,649	472,397	85.9	43,500	21,700	4.8	2,250,366
Utah	186,949	123,241	17.0	136,491	88,790	3.5	23,035	15,255	3.3	334,707
Vermont	40,708	18,109	1.2	180,204	59,279	7.8	46,514	23,257	1.1	88,372
Virginia	370,460	174,585	15.8	346,346	154,866	4.4	158,096	74,648	6.6	631,425
Washington	274,693	157,545	17.8	456,948	214,374	13.7				413,290
West Virginia	395,983	197,988	19.8	332,673	167,903	6.4				501,897
Wisconsin	935,930	468,059	42.7	1,382,342	634,463	46.0	76,438	37,300	.8	601,597
Wyoming	357,528	158,064	18.8	508,423	218,112	34.3				220,633
District of Columbia	80,772	39,924	0.9	2,558	1,279					161,433
Hawaii				2,375	2,375					334,875
Puerto Rico	105,633	51,430	6.4	125,732	61,425	4.2				221,235
TOTALS	26,425,273	12,967,308	1,782.1	27,033,690	13,572,401	1,280.1	9,004,798	5,189,126	617.9	30,614,086

Note: Includes apportionments for fiscal year 1943.

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