PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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PAVEMENT FAILURE CAUSED BY SLIDE IN SIDE FILL

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SUBGRADE SOIL CONSTANTS, THEIR SIGNIFICANCE, AND THEIR APPLICATION IN PRACTICE

Reported by C. A. HOGENTOGLER, Senior Highway Engineer, A. M. WINTERMYER, Assistant Highway Engineer, and E. A. WILLIS, Assistant Highway Engineer, United States Bureau of Public Roads

PART I: THE PHYSICAL PROPERTIES OF SOILS AND THEIR EFFECT ON SUBGRADE PERFORMANCE [1]

PRACTICAL information on the subject of subgrades is naturally divided into two classes: That which relates to the study of the soil, and that which relates to the practical utilization of the results of soil studies in the design and construction of roads. The former is of interest primarily to the subgrade testing engineer while the latter is of interest to every engineer who is now engaged in, and every engineering student who ever expects to engage in, the construction of works on the earth's surface.

The subgrade testing engineer who makes the studies must be familiar with the detailed procedures for making the subgrade surveys, the simplied subgrade soil tests, and the more elaborate Terzaghi tests. He must understand the complete significance of the various tests and the particular tests to be used for various purposes. The designing engineer requires only a superficial knowledge of the significance of the various tests and the procedures for making them. His special interest lies in the utilization of the test results to increase the stability and permanence of the structures with which he has to deal.

During the past few years the Bureau of Public Roads, by means of published reports, lectures, and exhibits, has made known to the interested public the progress of its subgrade studies. These researches have now reached a point where it is desirable to coordinate and summarize the results obtained.

The present report, of which the first part is published in this issue, consists of three major divisions: (1) A discussion of soil properties important with respect to subgrade performance, (2) the significance of the simplified soil tests for disclosing the presence of the important subgrade soil properties, and (3) the practical utilization of subgrade soil tests in practice.

The first division, which is intended primarily for the designing engineer and the engineering student, is included in Part I. In this part of the report the authors attempt in as simple a manner as possible to disclose the relation between the vehicle, the road, and the subgrade groups which have been suggested in a previous report and to discuss in a consistent order the various physical principles controlling the performance of the subgrade. An effort is made to show (*a*) that the subgrade instead of the pavement really supports the wheel load, (*b*) that the manner in which the subgrade supports the wheel load depends upon its reaction to both load and climatic changes, (*c*) that these reactions depend upon the five basic physical characteristics of soils, to wit, cohesion, internal friction, compressibility, elasticity, and capillarity, (*d*) that these physical characteristics control such important performances of subgrades as shrinkage, expansion, frost heave, the settlement of fills, sliding in cuts and lateral flow of soft undersoils, (*e*) that these physical characteristics are furnished by soil constituents easily identified in the laboratory and (*f*) that subgrades may be arranged in definite groups according to the characteristics of the soil constituents.

IMPORTANCE OF SUBGRADE SOIL CONSTANTS DISCUSSED

A subgrade soil test result may be defined as a measure of the degree in which a particular physical characteristic is exhibited when a soil is tested according to some arbitrary procedure. A subgrade soil constant may be either a test result as such or the result of a computation involving the use of several test results.

The subgrade soil constants to be employed beneficially in practice must serve to disclose the existence of those subgrade properties which exert an important influence upon the service rendered by road surfaces.

In order that subgrade soil constants may perform this service, one must have some conception of (*a*) those physical characteristics of subgrade soils which have an important bearing on the serviceability of road surfaces, (*b*) the influence exerted by the condition in which the soil exists and the character of its constituents upon the important subgrade soil properties, (*c*) the laws which control the physical characteristics possessed by subgrade soils, and (*d*) the degree to which subgrade soil constants disclose the presence of important subgrade characteristics.

Information of the character referred to is furnished by the subgrade investigations and the reports regarding them supplemented which have been published at different times in PUBLIC ROADS and elsewhere. These reports are listed in the bibliography included as part of this report.

While there is no intent to minimize in any manner the important influence exerted upon the properties of the soil by the state in which it exists, this report discusses primarily those properties characteristic of the raw constituents of soils regardless of state, and the importance of those properties with respect to road construction.

It should be remembered that the suggested subgrade groups are based upon subgrade performance. As additional information becomes available it might be desirable to subdivide certain of the groups with respect to the degree in which the subgrades possess particular properties, but the main groups are not likely to change in definition. The test constants which are being suggested as a means of identifying the members of the various groups are in a state of development and can not be considered as final. However, these constants and the scheme suggested for their use constitute the most logical method of soil identification yet disclosed by the bureau's subgrade investigations. This material is presented at this time not as a final and conclusive treatise on soil identification, but rather as a rational method by means of which the usefulness of test constants may be intelligently investigated.

Italic figures in parentheses ( ) used in this report refer to reports listed in the bibliography which furnish the material being discussed.

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[1] Parts II and III of this article will appear in the July, 1931, issue of Public Roads.

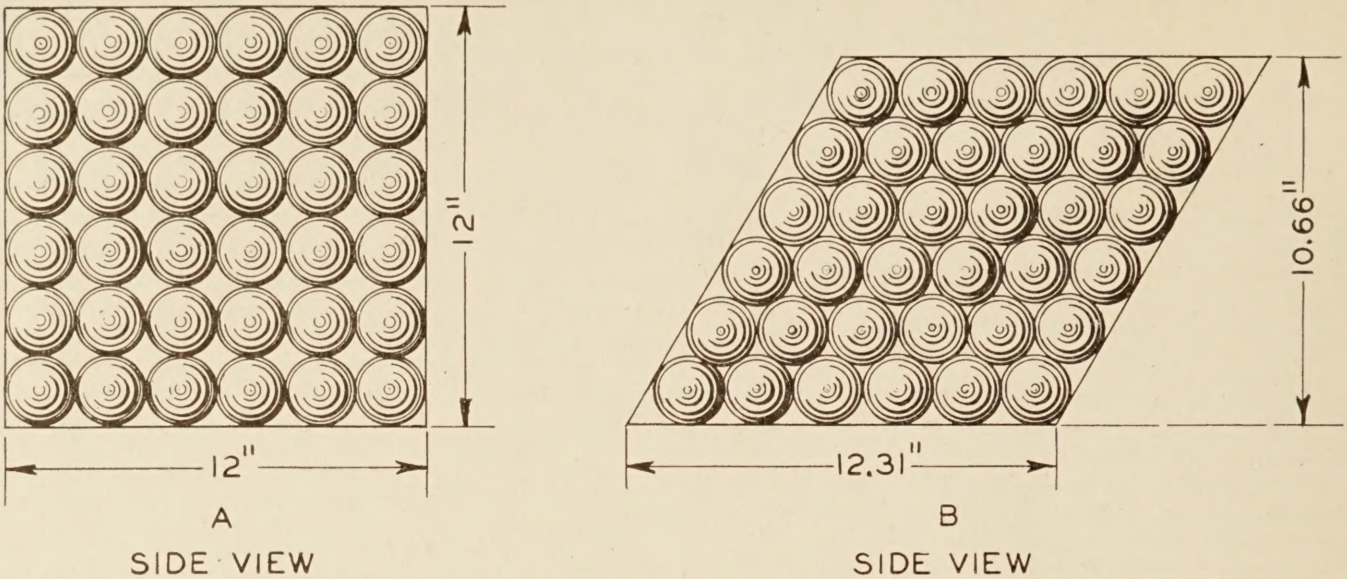

FIGURE 1.—SPHERES 2 INCHES IN DIAMETER ARRANGED IN LOOSE STATE, SIX POINTS OF CONTACT (A) AND IN MORE CONSOLIDATED STATE, EIGHT POINTS OF CONTACT (B)

Procedures for determining the different constants and for making subgrade surveys and mapping subgrade soil profiles are being prepared for publication at a later date.

BOTH SUPPORTING VALUE AND MOVEMENT OF SUBGRADE IMPORTANT IN ROAD CONSTRUCTION

In order to appreciate the importance of the subgrade in road construction one has only to remember that instead of the road surface, the ground or subgrade beneath really supports his pleasure car, truck, or other vehicle. The road surface or pavement merely distributes the vehicle weight over areas larger than those furnished by the tires alone.

Our high-class pavements, even the most substantial, when robbed of this ground support for appreciable distances, say 15 or 18 feet, break of their own weight and when unsupported for even short distances they become unable to withstand the weight of motor cars.

Thus the road surface furnishes only a smooth top on the natural ground surface and in order to accomplish this purpose two conditions must be fulfilled: The road surface must distribute the weight or impact delivered by motor vehicle wheels over an area sufficient to prevent appreciable depression of the ground beneath the pavement, and the ground beneath the pavement must be prevented from moving an amount sufficient to deform the road surface seriously. Otherwise, the road surface will fail.

When designing a highway the engineer is called upon to furnish a structure which, first, will resist the wear and tear caused by fast-moving motor wheels, second, will distribute wheel weights and impacts so as to prevent deformations which would be detrimental to either the road surface or the subgrade and, last, will resist natural forces to such an extent that their effect as manifested through subgrade movement will not be detrimental. In order to do this most economically, he must be cognizant of (1) the wheel loads and impacts to be resisted, (2) the relative ability of pavements to spread or distribute wheel loads, (3) the safe load the subgrade will support without depressing a detrimental amount, and (4) the movements likely to occur in the subgrade due either to climatic influences or to other causes. Only then is the engineer in a position to transform his road appropriations into the greatest mileage of serviceable highways.

It becomes evident, therefore, that in addition to studies of traffic weights and intensities and of pavement properties, none of which are discussed in this report, it is of great importance to investigate both the load-carrying properties of the subgrade and those soil characteristics which control subgrade movements other than those caused by vehicular loads.

VOIDS RATIO, VOLUME CHANGE, MOISTURE CONTENT, AND POROSITY EXPLAINED

It is necessary at this point to define certain terms, with full explanations of their significance. While the engineer who seeks only a general knowledge of subgrades may never have to use them in tests of his own, it is essential to an understanding of the subject that he know the precise meaning of the terms "voids ratio," "volume change," "moisture content," and "porosity," which have to do with those changes in soil state that affect the performance of subgrades.

A soil mass, or soil, as generally termed, consists of both soil particles and pores. When a soil mass, due to change in either moisture content or degree of consolidation, either increases or decreases in volume, only the void volume or the pore space is assumed to change, the volume of the soil particles remaining constant.

The density which controls in a large measure the supporting value of the soil depends upon the ratio of pore volume to either soil particle or soil mass volume. The test constants which represent either the moisture contents of soils when in particular states, or the changes in moisture content caused by changes in soil states, indicate, among other things, the density of the soil.

In order to visualize the soil states indicated by the constants and by the different degrees of soil density one must thoroughly understand the significance of the terms "voids ratio," "moisture content," and "porosity," which disclose the relation of pore volume to soil particle volume in the soil mass.

Voids ratio.—This term is defined as the ratio of the volume of voids to the volume of soil particles in a soil mass (*1*), i. e., the volume of the voids or pores per unit volume of soil particles in a soil mass.

Thus if e = voids ratio;
V_v = volume of voids;
V_s = volume of soil particles;

$$e=\frac{V_v}{V_s} \qquad (1)$$

and $e+1$ = total volume of soil mass per unit volume of soil particles in the mass.

The voids ratio, e, varies with (1) variation in degree of compaction, the number of soil particles remaining constant, (2) increase or decrease in moisture content, the number of soil particles remaining constant, and (3) increase or decrease in total number of soil particles, the volume of the soil mass remaining constant.

To illustrate the significance of the voids ratio, assume 216 spheres to be arranged as shown in Figure 1, A in a container 1 cubic foot in volume and a rectangular parallelepiped in form.

The combined volume of the voids and the spheres representing that of the soil mass equals that of the container, 1,728 cubic inches.

The volume of the spheres is given by the equation,

$$V_s=\frac{4}{3}\pi r^3\times 216=904.8 \text{ cubic inches}$$

The volume of voids is the difference between these two volumes,

$$V_v=1728-904.8=823.2 \text{ cubic inches}$$

Hence the voids ratio,

$$e=\frac{823.2}{904.8}=0.910$$

By a rearrangement of the spheres the voids ratio may be changed. This is illustrated by placing the 216 spheres in a container which is an oblique parallelepiped in form, as shown in Figure 1, B.

In this case the volume of the container is equal to the product, $12\times 12.31\times 10.66=1{,}575$ cubic inches. $V_v=1{,}575-905=670$ cubic inches; and the voids ratio, $e=\frac{670}{905}=0.740$.

Volume change.—This term is defined as the change in the volume of a soil mass due to change in the state of consolidation of the soil particles. Volume change is expressed in percentage of the volume of the soil mass either before or after change in the state of consolidation.

Thus when a soil changes from one state of consolidation indicated by a volume equal to V_1 to a different state of consolidation indicated by a volume V_2, C_1 is the volume change in percentage of the volume of the soil, V_1; and C_2 is the volume change in percentage of the volume V_2. Thus

$$C_1=\frac{V_1-V_2}{V_1}\times 100$$

$$C_2=\frac{V_1-V_2}{V_2}\times 100$$

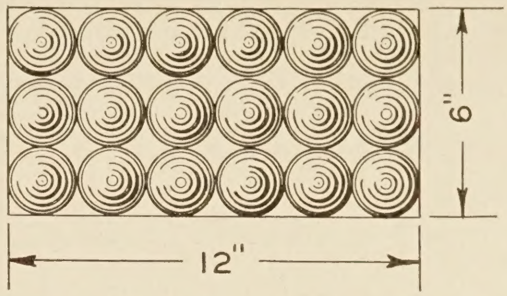

FIGURE 2.—108 SPHERES ARRANGED WITH SIX POINTS OF CONTACT ON EACH SPHERE, VOIDS RATIO EQUAL TO THAT OF THE 216 SPHERES SHOWN IN FIGURE 1, A

If the spheres shown in Figure 1, A are assumed to represent a state of consolidation indicated by V_1 and those shown in Figure 1, B are assumed to represent a state of consolidation indicated by V_2, the volume change obtained by changing the arrangement of the spheres from that shown in Figure 1, A to that shown in Figure 1, B is given by the formula,

$$C_2=\frac{V_1-V_2}{V_2}\times 100=\frac{1{,}728-1{,}575}{1{,}575}\times 100=9.7 \text{ per cent}$$

This method of computing volume change may be used whenever the volume of the soil mass but not the number of soil particles changes.

When a soil changes from any wet state indicated by a volume equal to V to the thoroughly dried state indicated by the volume equal to V_o, C is defined as the volume change in percentage of the volume of the soil in the wet state and C_o is defined as the volume change in percentage of the volume of the soil in the dry state. Thus

$$C=\frac{V-V_o}{V}\times 100; \text{ and}$$

$$C_o=\frac{V-V_o}{V_o}\times 100 \qquad (2)$$

Thus, for instance, the volume change which occurs when soil cakes in the laboratory are changed from the wet to the dry state, as in shrinkage tests, is given by this formula.

In case, however, one desires to compute the volume change when both the degree of consolidation and the number of soil particles change he must of necessity employ the voids ratio, which is expressed in unit values.

Assume, for instance, that the arrangement of the spheres shown in Figure 1, A remains the same but that the number of spheres is reduced to 108 as shown in Figure 2. The volume of the container is reduced by one-half but the voids ratio of necessity remains the same,

$$e=\frac{864-452.4}{452.4}=0.910$$

Let e_1 equal the voids ratio possessed by the 108 spheres shown in Figure 2, state 1, and e_2 equal the voids ratio of the 216 spheres shown in Figure 1, B, state 2. Then, if the degree of consolidation of the spheres is changed from state 1 (figs. 1, A or 2) to state

2 (fig. 1, B), the volume change of the container or soil mass per unit volume of soil particle, expressed in percentage of the volume of the container per unit volume of soil particle, state 2, is given by the expression,

$$C_2=\frac{(1+e_1)-(1+e_2)}{1+e_2}\times 100$$

$$=\frac{e_1-e_2}{1+e_2}\times 100 \qquad (3)$$

$$=\frac{0.910-0.740}{1+0.740}\times 100=9.8 \text{ per cent}$$

which substantially agrees with the value of C_2 obtained above, the difference being due to the lack of decimal places in the values of e.

Moisture content.—The moisture content, w, is defined as the weight of moisture in the soil in percentage of the weight of the soil particles.

Thus, if M_w is defined as the weight of the soil moisture in grams and W_o as the weight of the soil particles (weight of thoroughly dry sample) in grams,

$$w=\frac{M_w}{W_o}\times 100 \qquad (4)$$

To determine the moisture content possessed by a soil, the soil sample is weighed first wet and then dry. Hence, if W is defined as the weight of the wet sample (weight of soil particles + weight of moisture) and W_o as the weight of the dried soil sample,

$$M_w=W-W_o \qquad (5)$$

and

$$w=\frac{W-W_o}{W_o}\times 100 \qquad (6)$$

Because of the fact that 1 cubic centimeter of water weighs 1 gram, the weight of the water in grams, M_w, is also the volume of water in cubic centimeters. The volume of the soil particles in cubic centimeters equals the weight of the soil particles in grams divided by the specific gravity of the soil particles. Thus, if

V_s = volume of the soil particles in cubic centimeters;

and

G = specific gravity of the soil particles; then

$$V_s=\frac{W_o}{G} \qquad (7)$$

Consequently, w_v, the moisture content of the soil in percentage of the volume of the soil particles is given by the equation,

$$w_v=\frac{M_w}{\frac{W_o}{G}}\times 100$$

$$=\frac{W-W_o}{W_o}\times 100\times G$$

$$=wG \qquad (8)$$

Since w_v, the moisture content in percentage of the volume of the soil particles is equal to the void volume in percentage of the volume of soil particles, when the voids are completely filled with water, we have

$$w_v=e\times 100 \qquad (9)$$

and by substitution of wG for w_v,

$$e=\frac{wG}{100} \qquad (10)$$

Thus, if a soil sample weighs 30 grams when wet, 25 grams when dry, and the soil particles have a specific gravity of 2.5,

$$M_w=W-W_o=30-25=5 \text{ grams};$$

$$w=\frac{M_w}{W_o}\times 100=\frac{5\times 100}{25}=20 \text{ per cent};$$

$$w_v=w\times G=20\times 2.5=50 \text{ per cent};$$

$$e=\frac{w_v}{100}=0.5.$$

Porosity.—The porosity, P, is defined as the volume of the voids or pores in a soil mass in percentage of the volume of the soil mass (volume of soil particles + volume of the voids). Its value is given by the formula,

$$P=\frac{V_v}{V_v+V_s}\times 100=\frac{e}{1+e}\times 100 \qquad (11)$$

Thus, for the soil sample referred to above,

$$P=\frac{e}{1+e}\times 100=\frac{0.5\times 100}{1+0.5}=33.3 \text{ per cent.}$$

For the spheres shown in Figure 1, A,

$$P=\frac{0.91}{1+0.91}\times 100=47.6 \text{ per cent}$$

and for the spheres shown in Figure 1, B,

$$P=\frac{0.740}{1+0.740}\times 100=\frac{74}{1.74}=42.5 \text{ per cent.}$$

It has been shown [2] that the porosity of spheres of equal size when in the densest possible state equals approximately 26 per cent and the corresponding voids ratio approximately 0.35.

INFORMATION FURNISHED BY SUPPORTING VALUE TESTS LIMITED IN SCOPE

A scrutiny of reports on experiments dealing with the subject discloses a diversity of opinion as to the best manner not only of measuring but also of expressing the magnitude or efficiency of the support furnished to the road by the subgrade.

Thus the "comparative bearing value" of soils studied in the early investigations of the Bureau of Public Roads (*2*), the "modulus of subgrade reaction" used by H. M. Westergaard (*3*) in his discussions, and the "consistency" of soils investigated by Charles Terzaghi (*1*, *4*) are all indicative of supporting value and yet differ widely in both significance and scope.

The nature of the comparative bearing value test and the type of information furnished by it are illustrated in Figure 3. It is, in brief, a simple load-deformation, or load-penetration test of a small soil sample held in a container. A curve is drawn which shows

[2] Taylor and Thompson, Concrete, Plain and Reinforced, second edition, 1912, pp. 168–170.

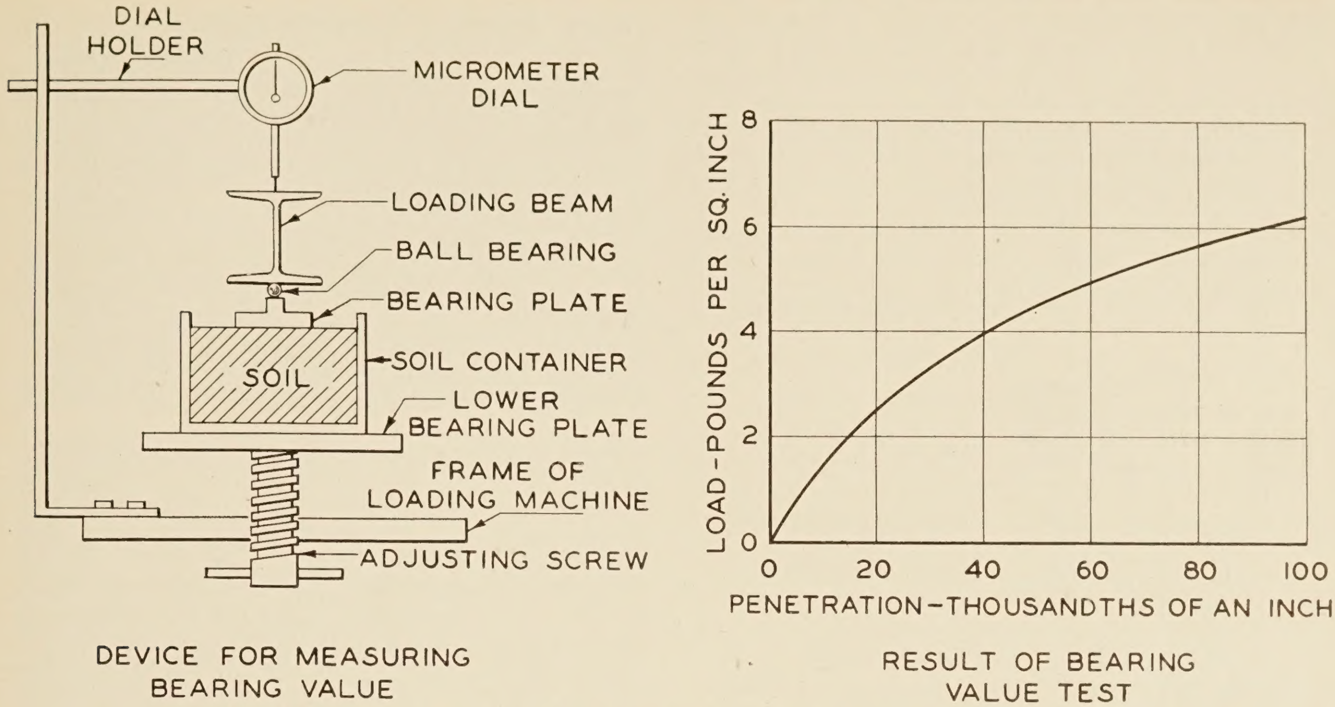

FIGURE 3.—APPARATUS AND TYPE OF RESULTS FURNISHED BY BEARING VALUE TEST

how much a given load will deform a given sample. Thus, according to Figure 3, a load of 4 pounds per square inch causes the bearing plate to penetrate a particular soil sample 0.041 inch. For comparative purposes, the bearing value of the soil was assumed to be the load in pounds per square inch required to produce a penetration of 0.1 inch.

Doctor Westergaard's modulus of subgrade reaction may be defined as the load in pounds per square inch required to deform a perfectly elastic subgrade 1 inch. Thus a perfectly uniform and perfectly elastic subgrade which will deform 0.01 inch for each one-half pound per square inch of pressure applied, has a modulus of subgrade reaction equal to 50.

The Terzaghi consistency test is performed on a cylindrical soil sample in the following manner. The sample is mounted in a loading machine equipped with a micrometer dial for measuring deformations, as shown in Figure 4. Load is applied slowly to a predetermined magnitude, and the sample is allowed to deform under this load until a state of equilibrium is reached. During this period of constant load the deformation is recorded as a function of the time. The load is then removed, and applied again to a greater magnitude. The deformation as a function of time is recorded for this load; the load is removed and applied a third time, until a point is reached (the yield point) where deformation is continuous without increase in load. The curve in Figure 4, center, shows the type of load-deformation curve which results from such a test. The curve at the bottom shows deformation plotted as a function of time.

It is evident that this test takes into consideration not only the load-deformation relation which the comparative bearing value test was designed to give, and the elastic rebound assumed by Westergaard, but also the effect of time on the deformation.

Even the consistency test, however, fails to supply complete information on subgrade support which should also include a knowledge of:

1. The deformation of the soil as influenced by (*a*) the magnitude of applied load, (*b*) the size and shape of the loaded area, and (*c*) a surcharge adjacent to the loaded area.

2. The relative amounts of the deformation due to (*a*) lateral displacement of the loaded soil and (*b*) compression of the under soil without lateral displacement.

3. The tendency of the soil to remain compressed or rebound upon the removal of load.

Direct bearing value tests, both in time and effort required, are generally too elaborate for use as routine tests for subgrade soils. Consequently, instead of direct tests of supporting value comparatively simple tests are used in the subgrade investigations to disclose the presence of subgrade characteristics indicative of three properties which either singly or in combination control the many types of deflection produced in soils by loading.

These properties af soils may be defined as follows:

1. Stability, the property of resisting lateral flow when loaded.

2. Compressibility, the property of compressing vertically under load without lateral movement and with a proportional decrease in air or moisture content.

3. Elasticity, the property of deforming under load and rebounding upon the removal of load without changing mositure content.

Figure 5, top, illustrates the character of deformation produced by loss of stability in soils. Here the load displaces the soil laterally. The deformation due to the compressibility of soils, illustrated in Figure 5, center, consists entirely of a more or less permanent consolidation of soil particles in the vertical direction. Figure 5, bottom, illustrates the rebound upon the removal of load in elastic soils.

Loss of stability may cause fills to slide, clay to work up into the interstices of base courses and rutting to occur in flexible road surfaces. Examples of loss of stability are illustrated in Figures 6 and 7.

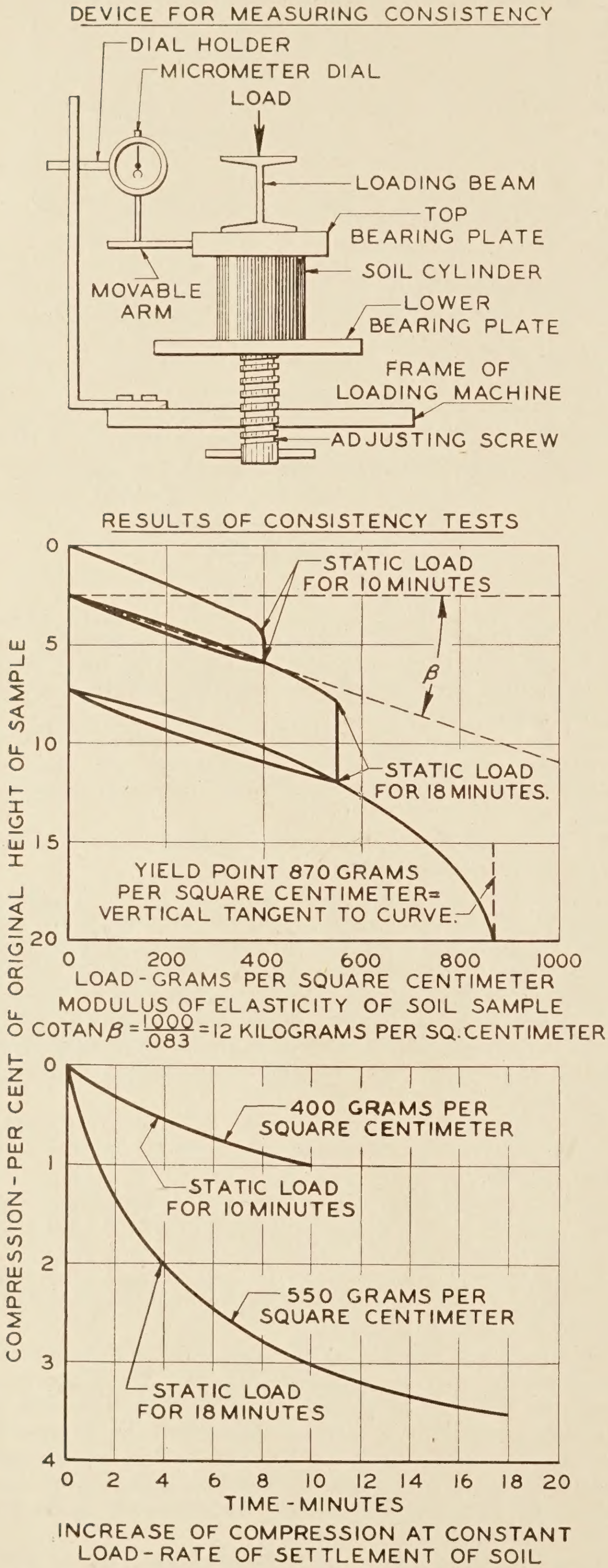

Figure 4.—Apparatus and Type of Results Furnished by Consistency Tests

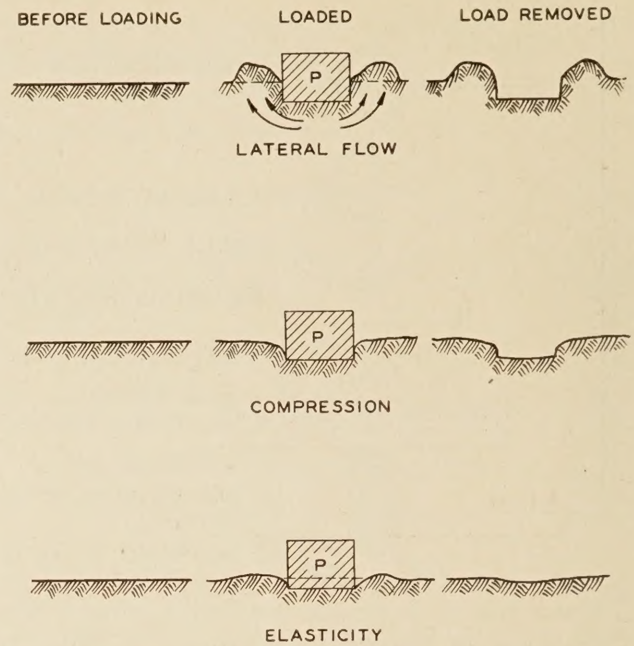

Figure 5.—Diagram Illustrating Properties on Which Reaction Between Soil and Load Depend

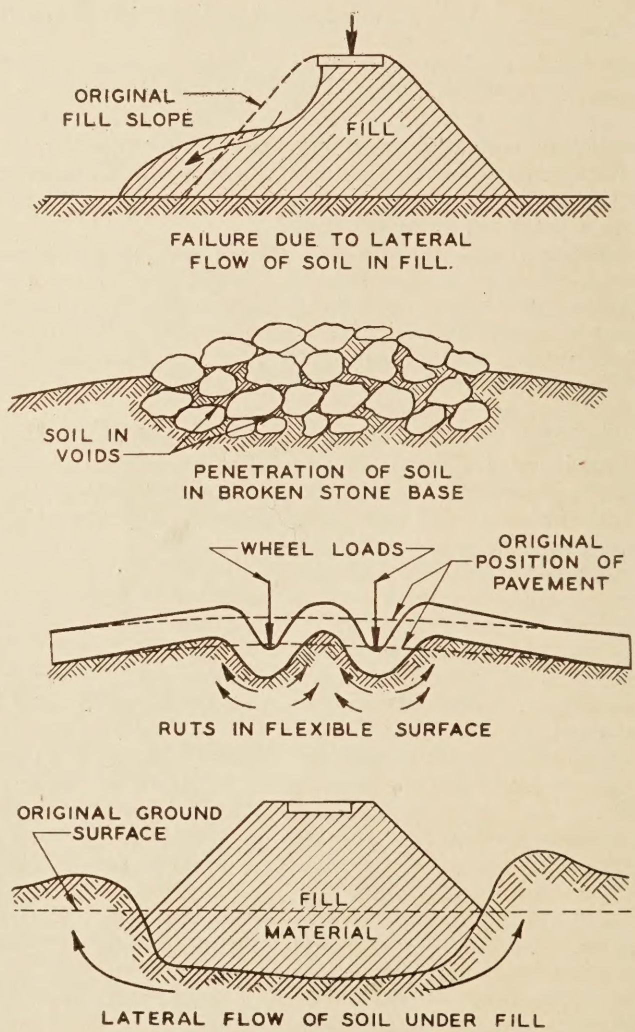

Figure 6.—Types of Road Failure Caused by Lateral Flow of the Subgrade Soil

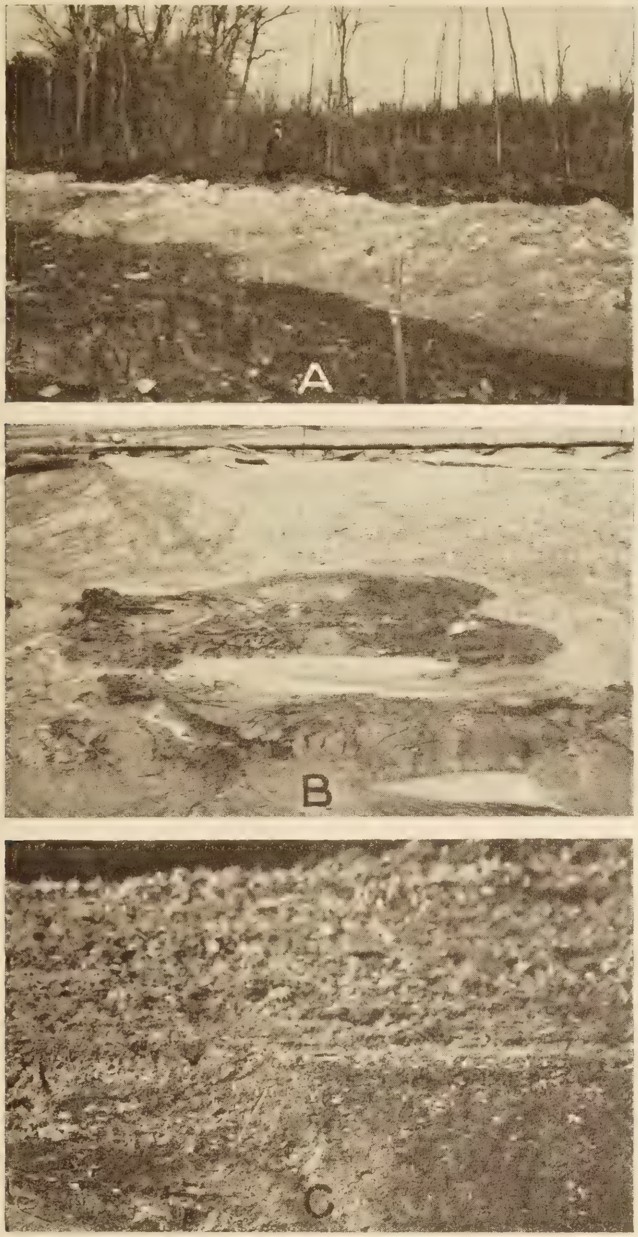

FIGURE 7.—EFFECTS OF LATERAL FLOW IN SOILS. A.—RIDGE OF MUCK SOIL FORCED UP BY LATERAL FLOW OF SOFT UNDERSOIL DUE TO WEIGHT OF FILL MATERIAL. DITCH OPENED AFTER UPHEAVAL. B.—RIVER BOTTOM FORCED UPWARD DUE TO LATERAL FLOW UNDER HYDRAULIC FILL. C.—SLIDE IN FACE OF CUT

STABILITY OF SOILS CONTROLLED BY THE COMBINED EFFECT OF INTERNAL FRICTION AND COHESION

Stability depends upon the shear strength which in turn depends upon the combined effect of the two mechanical properties of soils, internal friction and cohesion.

The magnitude of the cohesion possessed by a soil is independent of the outside pressure acting on the soil. It depends upon the stickiness of the soil grains or their resistance to being pulled apart and thus consists of the true cohesion of soil particles combined with that furnished by the molecular attraction of water (*5*, *6*). The stickiness of the clay in sand-clay roads and that of bituminous materials in black-top pavements represent true cohesion of materials. The very stable support furnished racing automobiles by beach sands when wet compared with the low stability of similar sands when dry serves to illustrate the importance of that portion of the total cohesion furnished by the molecular attration of water.

Internal friction, the magnitude of which increases in direct proportion to the pressure exerted upon the soil, depends upon the resistance of the soil grains to sliding over each other (*6*). It is defined as the angle whose tangent is the ratio between the resistance offered to sliding along any plane in the soil and the component of the applied force acting normal to that plane. The sand in sand-clay roads and the mineral aggregate in bituminous surfaces furnish the internal friction.

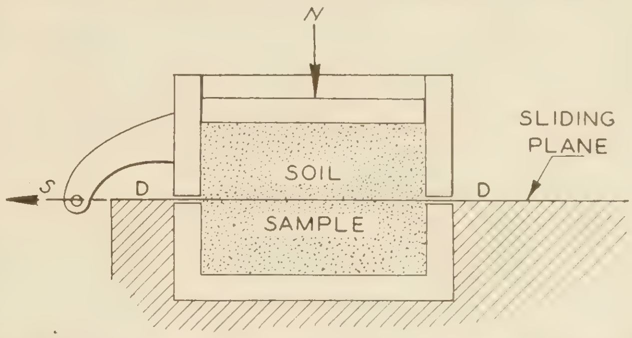

FIGURE 8.—SHEAR RESISTANCE OF SOILS AS RELATED TO INTERNAL FRICTION AND COHESION. BASIC PRINCIPLE OF APPARATUS TO DETERMINE MAGNITUDE OF THESE TWO MECHANICAL PROPERTIES

Figure 8 illustrates the influence exerted by both the cohesion and the internal friction upon the shear strength of soils.

The shear resistance is represented by the vector S. Let

N = Pressure acting on soil sample normal to the sliding plane;
c = Cohesion;
$= S$, when $N = o$;
ϕ = Angle of internal friction.

Then

$N \tan \phi$ = Frictional resistance to sliding;
$S = N \tan \phi + c$;

and

$$\phi = \text{arc tan } \frac{S-c}{N}.$$

Figure 9 illustrates the conditions required for stability in homogeneous soils.

Let DD be any plane in the soil making the angle α with the horizontal,

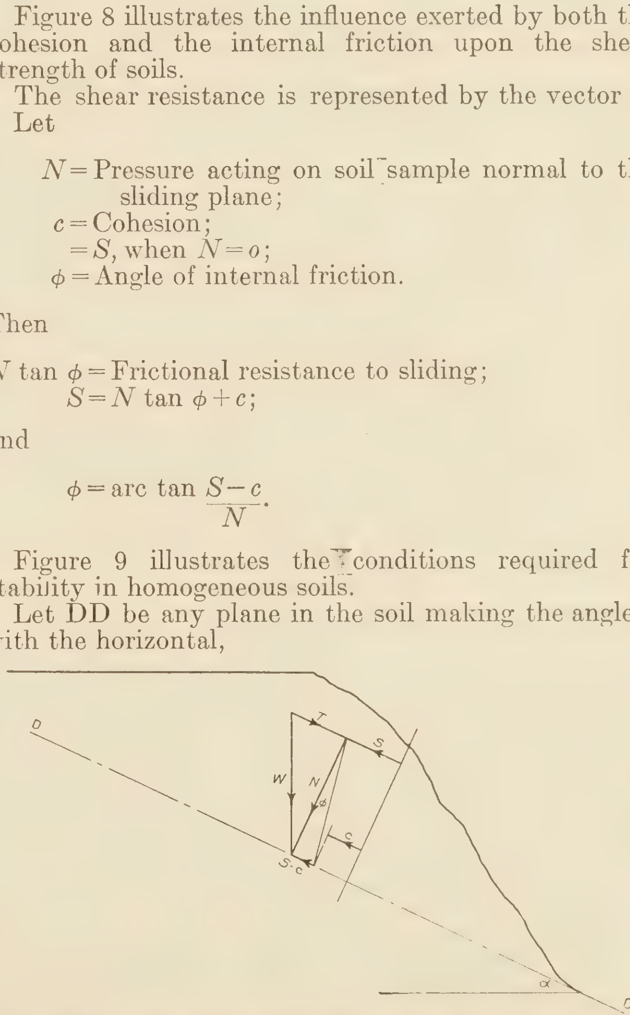

FIGURE 9.—THE MECHANICS OF SLIDING IN HOMOGENEOUS SOILS

W = unit weight of soil;
N = component of W normal to DD;
$= W \cos \alpha$;

then

T = force productive of sliding;
$= W \sin \alpha$;

and

S = shear resistance of soil;
$= N \tan \phi + c$.

Sliding occurs when T exceeds S. Therefore the requirement for stability is that $W \sin \alpha$ be less than $W \cos \alpha \tan \phi + c$.

Based upon this theoretical conception of stability, formulas have been developed by means of which may be determined (*a*) the influence of cohesion, internal friction, width of loaded area, and load adjacent to the loaded area upon the stability of subgrades (*7*) and (*b*) the influence of cohesion, internal friction and slope upon the critical height of fills (*8*).

The formula for computing the supporting value of soils was derived on the assumption that the loaded area was very long compared with its width. This assumption does not satisfy the condition produced by a wheel load upon the pavement, which is not susceptible of simple mathematical treatment. The analysis based upon a long loaded area illustrates the relative influence exerted by cohesion and internal friction upon the stability of soils.

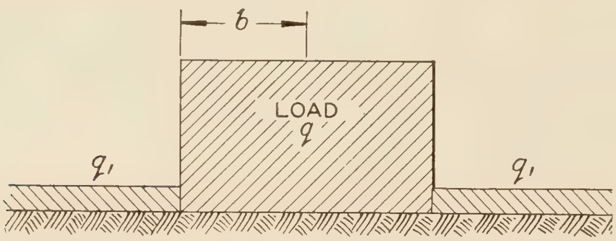

FIGURE 10.—DIAGRAM FOR ANALYSIS OF SUPPORTING POWER OF SOILS

Space does not permit derivation of the formula in this article (*7*). Figure 10 illustrates the method employed.

Let

q = load per unit area;
q_1 = surcharge adjacent to loaded area in same units;
b = ½ width of loaded area;
s = unit weight of soil;
c = cohesion (force per unit area);
ϕ = angle of internal friction;
$\beta = 45° - \frac{\phi}{2}$.

The load q, defined as the supporting value of the soil under the given conditions of cohesion, internal friction, width of loaded area, and surcharge, is given by the formula

$$q = \frac{q_1}{\tan^4\beta} + \frac{bs}{2\tan\beta}\left[\frac{1}{\tan^4\beta} - 1\right] + \frac{2c}{\tan\beta \sin^2\beta} \quad (12)$$

Résal's formula for computing the critical heights of cuts or fills (see fig. 11) gives only approximate results. It may be stated as follows:

Let

i = angle of inclination of fill;
ϕ = angle of internal friction;
Δ = unit weight of soil;
c = cohesion;

Then h_1, the critical height of fill above which sliding will occur is given by the formula,

$$h_1 = \frac{c \sin i \cos \phi}{\Delta \sin^2 \frac{i - \phi}{2}} \quad (13)$$

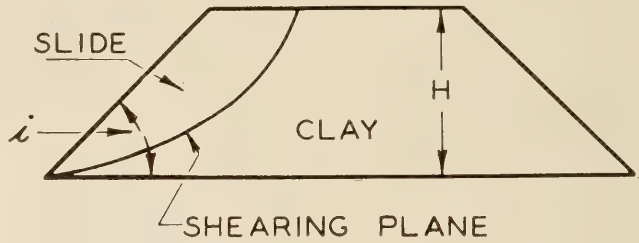

FIGURE 11.—DIAGRAM ILLUSTRATING RÉSAL'S FORMULA FOR COMPUTING THE CRITICAL HEIGHT OF CUTS OR FILLS

Results of computations made according to the formula of Figure 10 are shown in Table 1, and those made according to Résal's formula, Figure 11, are shown in Table 2.

TABLE 1.—*Influence of cohesion, internal friction, width of loaded area and load adjacent to loaded area upon stability of subgrades*

Soil type	Cohesion, c	Angle of internal friction, ϕ	Supporting value, q, pounds per square foot: $q_1=0$, $s=100$ pounds per cubic foot, $b=0.71$-foot	$q_1=100$, $s=100$ pounds per cubic foot, $b=0.71$-foot	$q_1=0$, $s=100$ pounds per cubic foot, $b=7.10$ feet
	Pounds per square foot	°			
Clay, almost liquid	100	0	400	500	400
Clay, very soft	200	2	860	980	910
Clay, soft	400	4	1,860	1,990	1,960
Clay, fairly stiff	1,000	6	4,980	5,130	5,170
Clay, very stiff	2,000	12	12,540	12,770	13,060
Silts,[1] wet	0	10	40	240	430
Sands, dry	0	34	770	2,020	7,680
Sand-gravel mixtures, cemented[2]	1,000	34	17,840	19,090	24,750

[1] In silty soils the angle of internal friction may vary between 10° and 30° but the cohesion may be almost 0.

[2] In properly graded soils, depending upon the extent of their compaction, the angle of internal friction may exceed 34° but the cohesion may be considerably less than 1,000.

TABLE 2.—*Critical heights of slope in cuts and fills, computed from Résal's formula*

Soil type	Slope of cut or fill	Angle of slope, i	Weight of soil, Δ	Cohesion of soil,[1] c	Angle of internal friction, ϕ	Critical height of fill, h_1
		° ′	*Pounds per cubic foot*	*Pounds per square foot*	*Degrees*	*Feet*
Very soft clay	½:1	63 26	80	200	2	9
	1:1	45 00				13
	2:1	26 34				25
	4:1	14 02				55
Medium clay	½:1	63 26	90	1,000	6	43
	1:1	45 00				70
	2:1	26 34				155
	4:1	14 02				546
Stiff clay	½:1	53 26	100	1,500	8	61
	1:1	45 00				104
	2:1	26 34				255
	4:1	14 02				1,300
Good sand clays	½:1	63 26	110	1,000	34	104
	1:1	45 00				580
	2:1	26 34				Unlimited.
	4:1	14 02				Unlimited.
Silty clays	½:1	63 26	100	200	14	10
	1:1	45 00				20
	2:1	26 34				72
	4:1	14 02				Unlimited.

[1] Materials having no cohesion such as sands, silts, etc., have no critical heights.

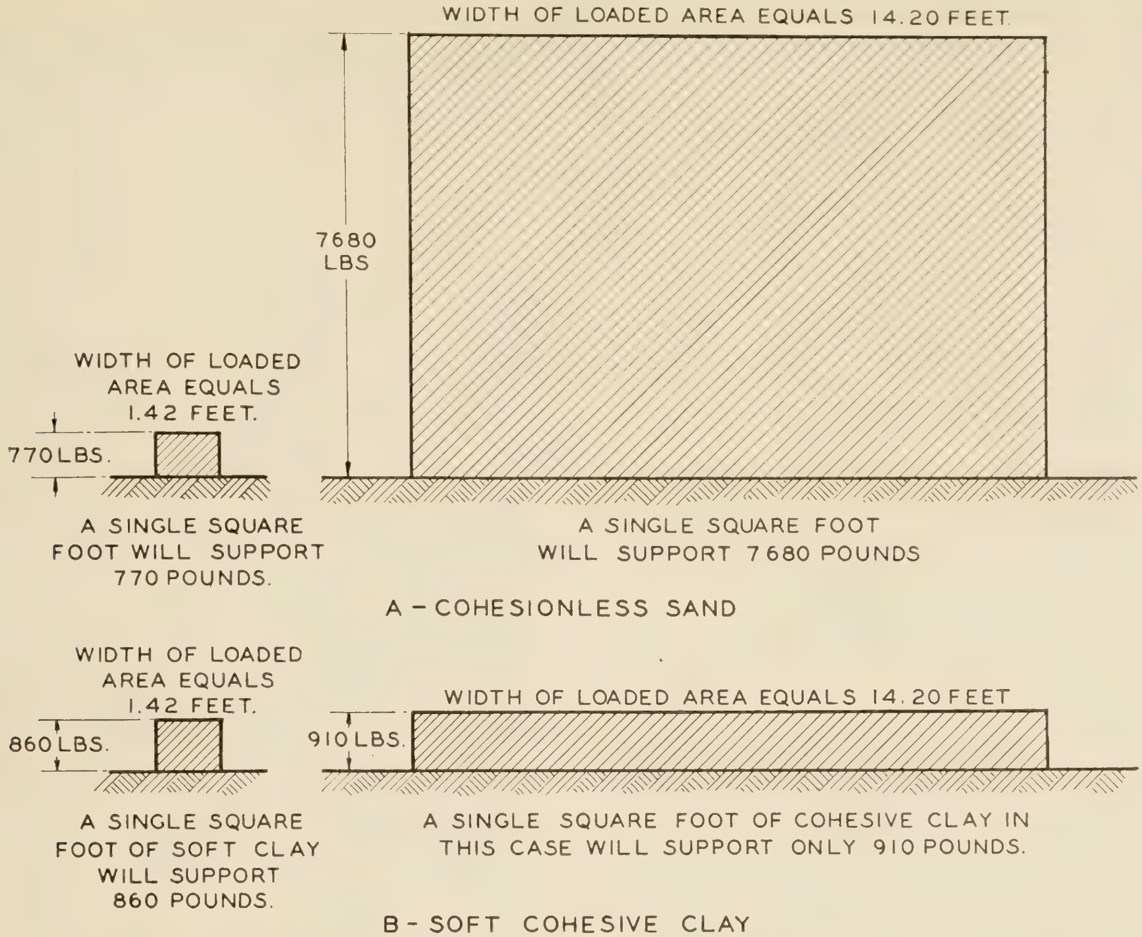

Figure 12.—Relative Effect on Supporting Power of Soils of Increasing the Area Under Load

These two formulas also serve to furnish some indication of the relative influence exerted on both the stability of subgrades and the critical height of fills by variations in the angle of internal friction, the cohesion remaining constant, or vice veras. Thus, when the cohesion equals 400 pounds per square foot, b equals 0.71 foot, and q_1 equals 0, the supporting value of the subgrade will equal either 2,550 or 7,600 pounds per square foot, depending on whether the angle of internal friction equals 12° or 34°. When the angle of internal friction equals 34° the supporting value of the subgrade will equal either 4,180 or 17,840 pounds per square foot, depending on whether the cohesion equals 200 or 1,000 pounds per square foot. Likewise, the critical height of fills with a slope of 1 to 1, a weight of soil, Δ, of 100 pounds per cubic foot, and a cohesion of 400 pounds per square foot will be either 21 or 34 feet, depending on whether the angle of internal friction equals 2° or 12°; and when the angle of internal friction equals 6° the critical height will equal 13 or 63 feet, depending on whether the cohesion equals 200 or 1,000 pounds per square foot.

These examples show not only that stability depends upon both the internal friction and the cohesion of the soil, but also that the manner in which stability is influenced by such factors as the size of the loaded area, etc., differs widely depending on whether the stability is furnished principally by internal friction or cohesion. Thus:

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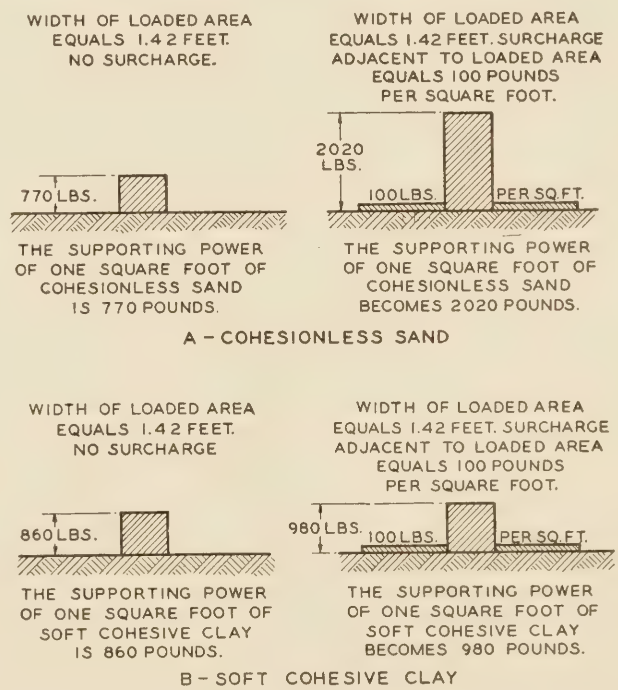


Figure 13.—Relative Effect on Supporting Power of Soils of Uniformly Loading the Soil Adjacent to the Loaded Area

1. Increasing the width of the loaded area and also surcharging the soil with a load adjacent to the loaded area increases the unit support very appreciably, as shown in Table 1 and Figures 12 and 13, when the stability of the subgrades is furnished principally by internal friction instead of cohesion. The increases in the unit support of cohesionless soils due to increase in width of loaded area and to the surcharging just noted are, according to unpublished data furnished by Dr. Charles Terzaghi, much greater when the loaded area is long and narrow than when it is square or circular.

2. Increasing the width of the loaded area or surcharging the soil with a load adjacent to the loaded area does not increase the unit support very appreciably when the stability of the subgrade is furnished principally by cohesion instead of internal friction.

3. The safe angle of repose of fill material is independent of the height of the fill only when the fill consists of cohesionless materials.

Thus one sees how a conception of the effect of the relative amounts of cohesion and internal friction possessed by the soil is more enlightening with respect to the design of preventive measures than merely a knowledge of stability or the combined effect of these two properties. Figure 14 shows a method of preventing sliding by terracing the faces of a cut made in clay.

Figure 14.—Terracing of Faces of Railway Cut in Clay, Between Washington and Baltimore. Surface Drainage is Provided for on Each Terrace. One of the Methods Used to Prevent Sliding in Cuts

RECOGNITION OF COMPRESSIBILITY AND ELASTICITY IN SUBGRADES IMPORTANT

Consolidating the subgrade serves to increase its density and decrease its permeability (*1, 7, 9, 10*)[3] and consequently is likely to prove highly beneficial. The degree of consolidation obtainable and the nature of the results, whether beneficial or detrimental, which will be obtained by attempted consolidation depend upon whether the subgrade soils are of the compressible or of the elastic type; that is, whether in the absence of change in moisture content they will remain consolidated or will rebound upon the removal of load. To remain consolidated after the removal of load, in the absence of free water, soil grains must either lack any spongy or elastic property which tends to push them apart or possess cohesion in amount sufficient to overcome such a tendency.

A small sponge and a wad of cotton will serve to illustrate very effectively how the elastic subgrades differ in performance from the compressible subgrades. If, for instance, they are both thoroughly wetted and then compressed by hand the cotton, representing the compressible soil, remains in the compressed state after the removal of the compressing force, because of capillary tension acting on the surface of the cotton—the same force which stabilized the beach sand referred to above. The capillary tension in this case equals at least the pressure exerted by the hand to compress the cotton and also any tendency possessed by the cotton strands to separate, otherwise the cotton would have expanded. The sponge, representing the elastic subgrades, expands to almost its wet volume upon the removal of the compressing force. This occurs because the tendency of the sponge fabric to expand greatly exceeds the capillary force acting upon the sponge surface. Soaking the sponge prior to compression in a glue whose molecular cohesion exceeds that of water in sufficient amount serves to prevent the elastic rebound upon the removal of pressure.

Figure 15.—Detrimental Effects of Elasticity of Subgrade Soil. A.—Type of Cracking Likely to Occur in Pavements Laid on Improperly Prepared Elastic Subgrades. Note Cracks Reflecting Ruts in Subgrade. B.—Alligator Hide Cracking in Macadam Road Surface

Either a macadam or a concrete surface may be seriously damaged by attempts to consolidate elastic subgrades before pavement construction. After the thorough rolling which benefits the compressible subgrades, a subgrade of the elastic type, if it possesses cohesion, is likely to retain a certain degree of compaction. A slight wetting under these conditions, such as is furnished by freshly deposited concrete, may cause a nonuniform rebound of the subgrade. This, combined with water loss from the concrete due to absorption by the soil is likely to cause pavements to crack excessively (fig. 15, A) during the setting period of the concrete.

Movements of heavy material trucks and mixing apparatus adjacent to pavements laid on elastic co-

[3] Permeability is defined as the rate at which gravitational water is transmitted by soils. It depends upon both the hydraulic gradient and the size and number of the soil pores. It varies as the square of the effective diameter of the soil grains. It is expressed as the coefficient of permeability which is designated as k and equals the velocity in centimeters per second under a hydraulic gradient of 1.

hesionless subgrades may cause distortions of the soil supporting the freshly laid concrete sufficient to produce pavement cracking. Cracking of this character may remain in microscopic form for an appreciable period of time. Except during the setting period of the concrete, elastic subgrades are not likely to be detrimental to concrete pavements.

The presence of elasticity in subgrades may prevent macadam pavements from acquiring adequate bond during construction and from retaining it subsequently. Under these conditions macadams may develop "alligator hide" (fig. 15, B) cracking, through which water may pass and cause the subgrade soil to soften and to penetrate the voids of the macadam, thus causing the surface to fail.

Figure 16.—Fill Being Constructed of Material Consisting of Clods

DEGREE AND RATE OF COMPRESSION DISCUSSED

The manner in which soils may compress depends to a large extent upon their moisture contents. Those soils whose voids contain air may compress because of either a compression of the entrapped air or the escape of the air from the soil pores. In this case the rate of consolidation depends upon such factors as the resistance of clods (fig. 16) to crushing and can not be computed. The force required to break clods differing in degree of dryness can, however, be investigated in the laboratory.

Soils in the plastic state (fig. 17) or those whose voids are filled with moisture, may consolidate vertically without flowing laterally only when water escapes from the soil pores. Thus foundations supporting buildings and other structures adequately for years settle suddenly when new excavations permit water to escape from the loaded soil supporting the foundations.

In this case the speed of soil consolidation for equal external pressures applied depends primarily upon the permeability of the soil mass. In fact within certain limits it varies directly with the coefficient of permeability of the soil. Since the coefficients of permeability of the soil constituents may vary through a wide range, as shown in Table 3, the rates at which individual soils consolidate may be widely different.

Figure 18 shows how the data furnished by the Terzaghi compression test may be employed to indicate both the amount and the rate of fill settlement. This test has been discussed previously in Public Roads by Dr. Charles Terzaghi (*1*).

Water pressed from the soil pat (fig. 18, A) by the weighted piston, passes through the porous stones above and below the pat and escapes from the over-

Table 3.—*Coefficients of permeability of soil constituents under pressure of 1.5 kilograms per square centimeter*

Soil	Coefficient of permeability
	Centimeter per second
Potomac River sand, 20–100 mesh	18.96×10^{-4}
Mica, 20–100 mesh	0.128×10^{-4}
Rock Creek silt	0.00096×10^{-4}
Diatoms	0.048×10^{-4}
Clay	0.00011×10^{-4}
Peat (Minn.)	0.785×10^{-4}

flow orifices *a* and *b*. The relation between the voids ratio of the soil and the compressing force is expressed as the load-compression curve, Figure 18, B. The data for constructing the load-compression curve are obtained by applying the compressing force in magnitudes equal approximately to 0, 0.5, 1.5, and 3.0 kilograms per square centimeter and observing the voids ratio produced by each load when applied, until further increase in the deformation of the soil ceases. Consequently the load-compression curve discloses the minimum voids ratio or the maximum density of the soil likely to be produced by loads of given magnitude. Data for constructing the expansion curve (fig. 18, B) are obtained when the load is changed successively from approximately 3.0 to 1.5, 0.5, and 0.0 kilograms

Figure 17.—Settlement of Road at Bridge Approach Due Primarily to the Consolidation of the Thoroughly Saturated Undersoil

per square centimeter and water is allowed to enter the sample. The data for constructing the time-compression curve are furnished by observing the times corresponding to the deformations produced by the individual load increments.

The time-compression curve (fig. 18, C) shows the relation between (*a*) the degree of compression of the soil, expressed as a percentage of the total compression occurring in a very long loaded interval, and (*b*) the length of the loaded interval expressed in minutes, the magnitude of the load remaining constant.

In order to demonstrate, by analogy, the method of estimating both the magnitude and the rate of compression caused by a hydraulic fill constructed on inundated river bottom land as illustrated in Figure 18, D, it must be assumed first that the compression is direct and no lateral thrust is involved; second, that both the fill material on top and the hard compact sand beneath are more permeable than the soft soil layer; third, that the moisture is free to pass through the underlayer of

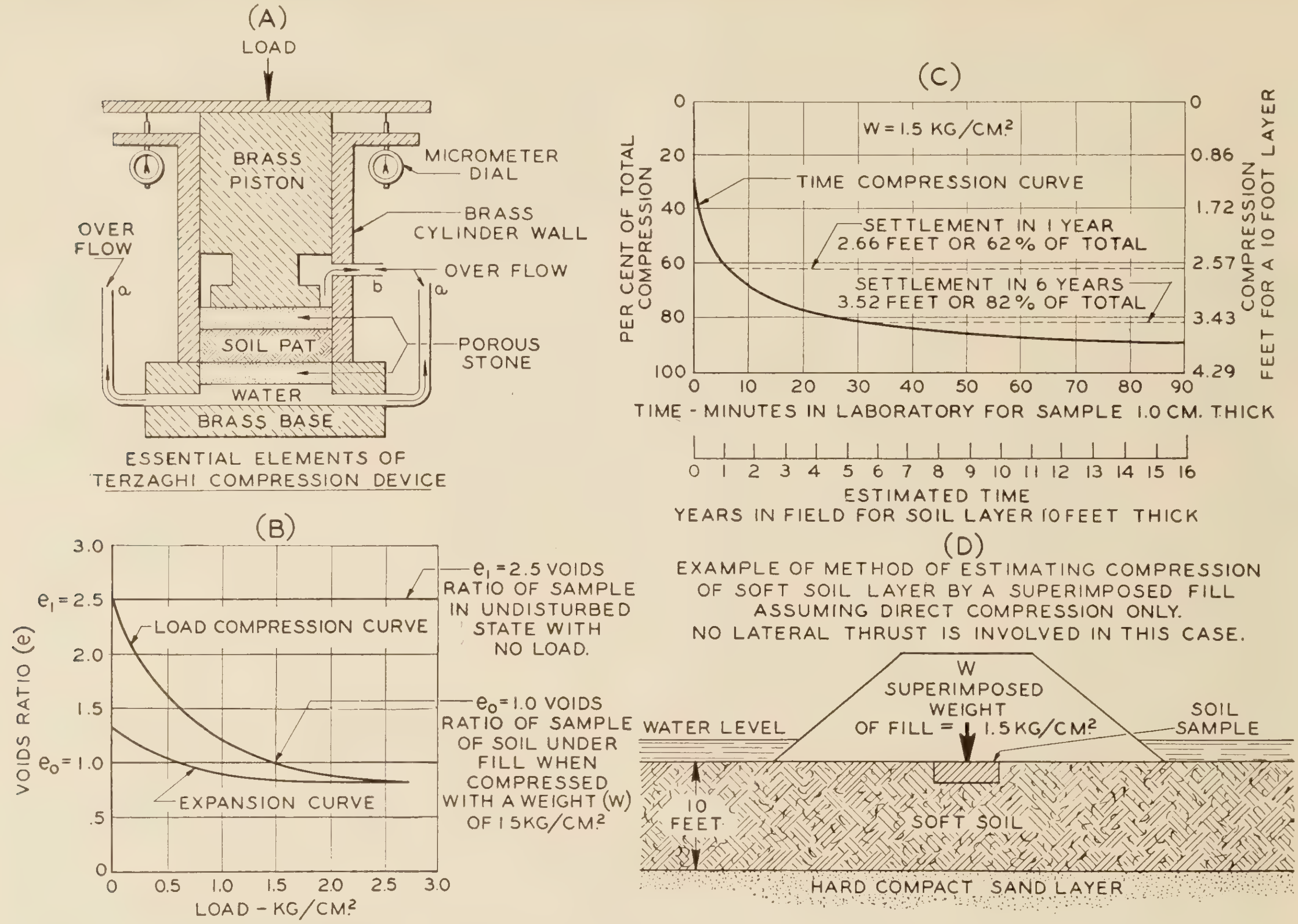


Figure 18.—Data Furnished by the Terzaghi Compression Test

sand, and last, that the moisture content throughout the thickness of the soft soil layer is uniform.

Under these conditions the soft soil layer in the river bottom (fig. 18, D) is subjected to compression similar to that acting on the soil pat (fig. 18, A). Thus the weight of the fill material corresponds to the compressing force, and the fill material on top and the compact sand beneath the soft soil layer correspond to the porous stones. (Fig. 18, A). In addition let the load-compression curve (fig. 18, B) and the time-compression curve (fig. 18, C) be assumed to represent tests performed upon an undisturbed sample of the soft river bottom soil, as indicated in Figure 18, D.

Under the weight of the fill material (1½ kilograms per square centimeter), the undersoil will, since the assumed conditions are the same as those existing in the laboratory, compress from a voids ratio $e_1 = 2.5$ (original undisturbed state) to a voids ratio $e_0 = 1$. The ratio of soil thickness after compression to soil thickness before compression equals the ratio of soil volume (soil particles plus voids) after compression to the soil volume before compression, or $\frac{1+e_0}{1+e_1}$. Consequently the soil layer 10 feet thick will compress to a layer whose thickness is given by the expression

$$\frac{1+e_0}{1+e_1} \times 10 = 5.71 \text{ feet.}$$

The time in years required for different stages of settlement is computed from the time-compression curve on the assumption that the time required for two soil layers to compress in equal degree varies as the squares of the thickness of the layers.

Accordingly Figure 18 informs us (*a*) that the fill will settle 4.29 feet and (*b*) that 2.66 feet of this amount will occur during the first year and that an additional settlement of 0.86 foot will occur during the succeeding five years.

CAPILLARITY THE IMPORTANT AGENT CAUSING CHANGES OF WEATHER TO BE REFLECTED IN SUBGRADE MOVEMENTS

The more important subgrade movements due to climatic influences are (*a*) expansion of the soil occurring with an increase in moisture content (*b*) shrinkage of the soil occurring with a decrease in moisture content, and (*c*) heaving of the soil during frost.

These occurrences depend upon the physical phenomenon capillarity, which, so far as subgrades are concerned, is defined as the ability of soils to transmit moisture in a finely divided state in all directions in spite of both the direction in which gravity acts and the force of gravity.

Capillary action is illustrated in Figure 19. Capillarity draws the liquid up through the cheesecloth wicks, over the edge of the container A, and down to the ends of the wicks on the outside of the container. This merely moistens the wick on the left whose outside end is just about on a level with the surface of the liquid in the container. In addition to becoming moist, the wick on the right, the outside end of which is located appreciably

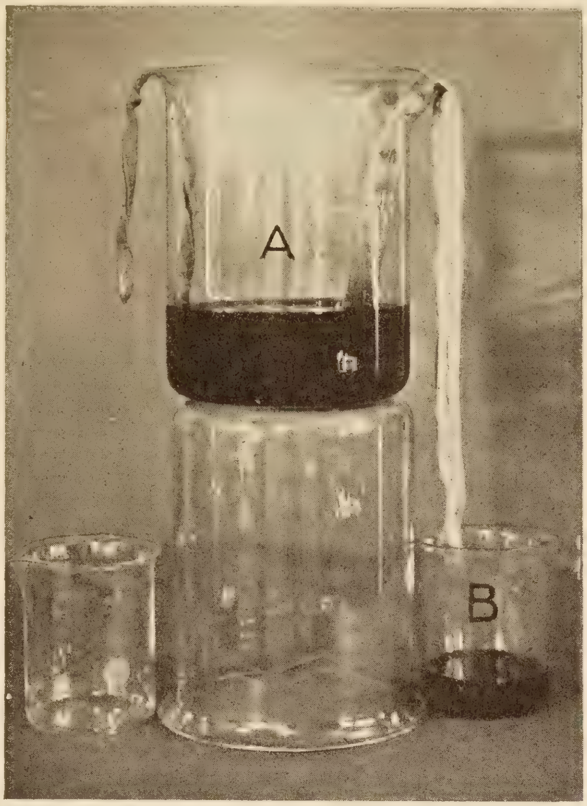


FIGURE 19.—ILLUSTRATION OF CAPILLARITY

below the surface of the liquid in the container, performs like a syphon and transfers the liquid from container A to container B.

When this photograph was taken the liquid was dropping into container B from the end of the cheesecloth wick at the rate of 1 drop every 38 seconds or

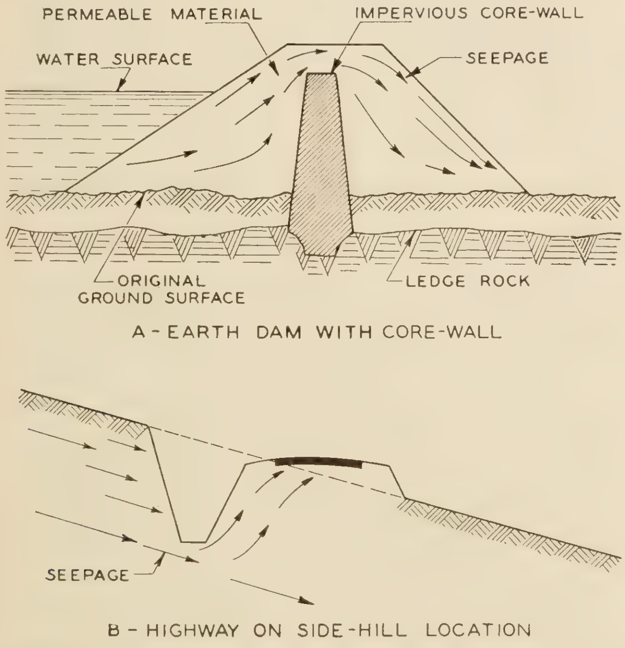


FIGURE 20.—ILLUSTRATIONS OF CAPILLARY ACTION IN SOILS

1 gram every 10 minutes. This equals 1 gallon every 631 hours. At times during the experiment the rate was as much as 1 gallon every 12 days.

This simple experiment explains the occurrence of seepage in the lower face of an earth dam, even though the top of the impervious corewall is higher than the adjacent water surface, Figure 20, A, and in subgrades in spite of ditches, as shown in Figure 20, B.

The maximum distance through which water may be forced by capillarity depends upon the surface tension of the water and the size of the soil pores, and increases as the size of the soil pores decreases, the temperature of the water remaining constant (*1*, *5*). The rate at which capillary moisture travels depends upon the capillary tension, upon the frictional resistance furnished by the walls of the pores to the flow of water, and upon the rate at which capillary equilibrium is destroyed by evaporation, the formation of ice crystals, or change in ground water elevation (*11*).

FIGURE 21.—SOIL IN FACE OF CUT ERODED BECAUSE OF ITS EXPANSIVE PROPERTIES

PROPERTIES AFFECTING EXPANSION AND SHRINKAGE OF SOILS ANALYZED

The extent to which water will be absorbed depends upon both the capillary properties and the degree of cohesion possessed by the soil. Water entering cohesionless soils through capillary action may cause the grains to separate to such an extent that the soil quickly disintegrates, as shown in Figure 21. A sufficient amount of cohesion existing between the soil particles will prevent the entrance of water in an amount sufficient to cause the soil to lose stability, unless the soil is manipulated. The amount of cohesion possessed by a soil of given constituents depends upon both the moisture content and the state of compaction of the soil. Therefore the relative amount to which the soil will expand depends upon both the degree of consolidation and the moisture content of the soil before wetting.

Each of 90 soil cakes were compressed in the wet state in the subgrade laboratory of the Bureau of

Public Roads, under a load of 3 kilograms per square centimeter. The load was then reduced to 0.028 kilogram per square centimeter and the cakes were permitted to absorb water. Subsequently two disks, each 1 square inch in area, were cut from each of the 90 soil cakes. One of these disks, in the wet state, was immersed in water and its counterpart was first allowed to dry to constant weight in the air and was then immersed in water.

Eight of the 90 disks, containing less than 12 per cent clay and immersed in the wet state disintegrated after being immersed for periods averaging 73 days. Of the 8 corresponding disks immersed in the dry state, 6 disintegrated after being immersed for an average period of 7 minutes and the remaining 2 swelled and cracked in appreciable amount but did not completely disintegrate after being immersed for a period of 25 days. Sixty-eight of the 90 disks containing, with several exceptions, clay 13 to 77 per cent and immersed in the wet state, remained intact after being immersed for an average period of nine months. Of the corresponding 68 disks immersed in the dry state, 26 disintegrated after being immersed for an average period of 10 minutes, 41 disintegrated after being immersed for an average period of 1 hour, and the remaining disk cracked and swelled in appreciable amount after being immersed for a period of 10 days.

As additional evidence that cohesion in soils tends to prevent their expansion and disintegration due to water absorption reference is made to Figure 22. The soil cakes shown in this figure were made up from soil samples taken at different depths in a subgrade located at Arlington, Va., which was treated with water-gas tar in 1923. After their removal from the subgrade in 1929, these cakes were compressed in a semidry state, dried to constant weight in the air, and immersed in water for a period of two weeks. The two cakes shown on the left of Figure 22 contain tar in appreciable amount and exhibit no signs of disintegration. The third cake from the left contains but a small amount of tar and has crumbled slightly along the top edges. The fourth cake from the left contains but a slight trace of tar and shows crumbling in appreciable amount near the top. The two cakes on the right contain no visible trace of tar and have crumbled to a still greater extent.

Figure 22.—Soil Samples Containing Water-Gas Tar in Varying Amounts, Photographed After Immersion in Water for a Period of Two Weeks

Reduction in soil volume as illustrated in Figure 23 is caused by reduction in the moisture content of the soil. This action depends upon the capillary force exerted as the moisture content is reduced by evaporation and upon the resistance furnished by the soil particles to being consolidated. The theory of shrinkage in soils may be briefly stated as follows (*12*): Assume a soil with its pores completely filled with moisture to be in the liquid state. Under these conditions the contractive force exerted by the surface tension of the water is practically zero. As water evaporates from the sample the capillary tension exerts on the outer of surface of the sample a uniformly distributed force acting at every point perpendicular to the outer surface of the sample, and tending to draw the particles in. As the soil sample becomes smaller and smaller its resistance to further shrinkage correspondingly in-

Figure 23.—Large Shrinkage Cracks Over 3 Inches Wide and Over 1 Foot Deep

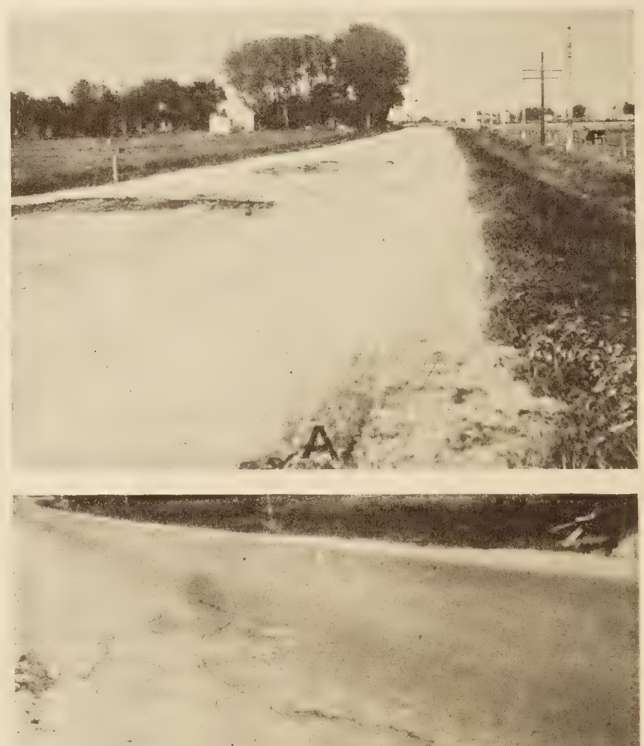
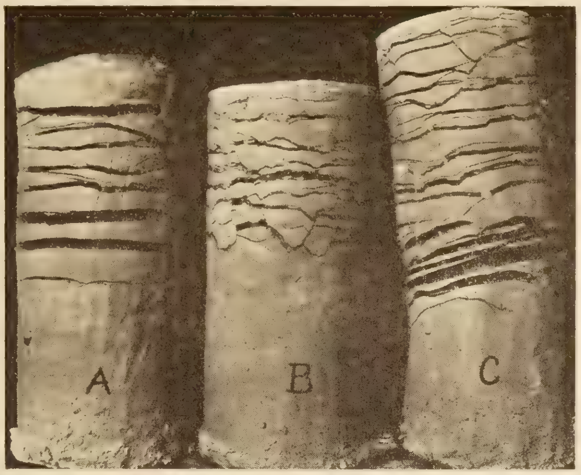


Figure 24.—Examples of Frost Damage in Roads. A.—Gravel Road Heaved by Frost Action. B.—Bituminous Macadam Road Damaged by Frost Action in Subgrade



Ice lenses in test cylinders frozen by Prof. Stephen Taber

Ice lenses observed by F. C. Lang in soil in Minnesota

FIGURE 25.—EXAMPLES OF ICE SEGREGATION IN SOILS

creases. Finally the soil attains a volume at which the resistance of the soil sample to further reduction in volume just equals the capillary pressure exerted by the evaporating moisture. Further evaporation will not appreciably decrease the volume of the soil sample. The moisture content of the soil at this state of equilibrium is termed "the shrinkage limit" and is discussed in Part II of this report, to be published in the next issue of PUBLIC ROADS.

DETRIMENTAL HEAVING CAUSED BY SEGREGATION OF WATER WHEN FREEZING

Heaving of soils due to frost action (fig. 24, A and B) is caused by an increase in total moisture content occurring as ice layers or crystals (*13, 14, 15*). Its magnitude depends upon the rate of temperature change, the moisture content of the soil prior to freezing, the proximity of additional water and the rate at which capillary flow occurs (*13, 14, 15, 16*).

The formation of well defined ice layers in the soil (figs. 25,[4] and 26) depends on three physical phenomena—

(*a*) The tendency of water particles contained in soil pores of the larger capillary dimensions to freeze at either normal freezing or slightly less than normal freezing temperatures (−1°, −4° C.).

(*b*) The tendency of water particles contained in soil pores of the smaller capillary dimensions to resist freezing at abnormally low temperatures (as low as −70° C.).

(*c*) The tendency of water particles of freezable size, during the process of freezing, to draw to themselves from adjacent fine capillaries the small particles of water which individually do not freeze at ordinary freezing temperatures (*14*). When drawn to the existing ice crystal these small water particles freeze and increase in size. Continuation of this process causes the original ice crystals to increase in size as long as they are being supplied with small water particles drawn up through the fine capillaries from the ground water supply.

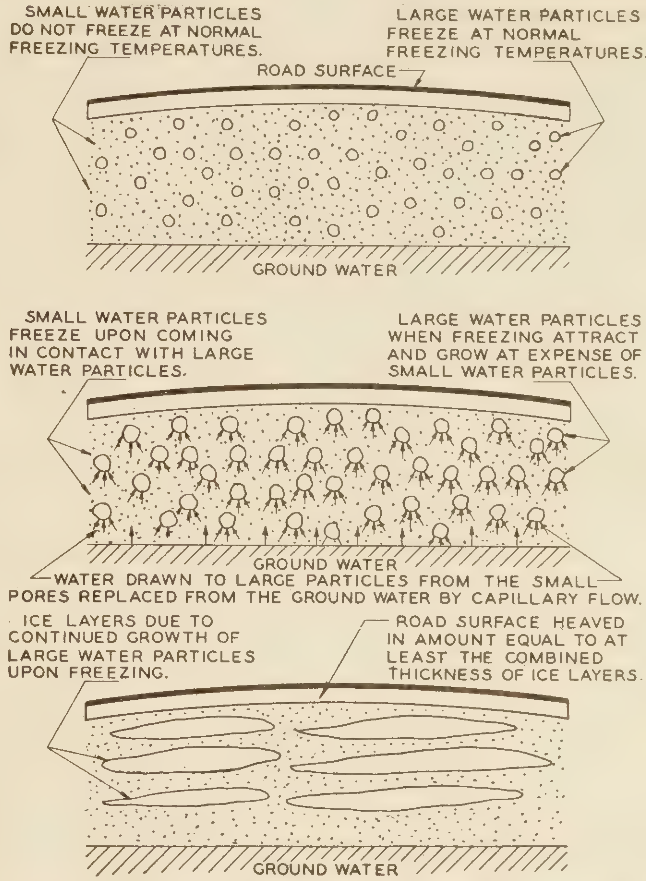


FIGURE 26.—DIAGRAM ILLUSTRATING THE PHYSICS OF FROST HEAVE

Whether or not frost heave will occur depends upon the quantity of moisture capable of being raised to a given height above the water table in a given time. Neither the height to which water will rise by capillarity nor the rate of such rise is alone the determining factor. Just as the amount of water furnished by a pipe depends upon the pressure acting on the water, the diameter of the pipe and the frictional resistance to flow, the raising of a quantity of water sufficient to produce frost heave in the subgrade at a given height above the ground water elevation (disregarding the rate at which capillary equilibrium is destroyed) depends upon the force of capillarity, the area of pore space, and the frictional resistance.

Capillary pressure varies inversely with the diameter of the pores. The frictional resistance to flow through a soil is a function of the surface area of the soil particles and consequently increases with a decrease in grain size at a much greater rate than does the capillary pressure. Therefore, in order to furnish capillary

[4] Reproduced from the article, Freezing and Thawing of Soils as Factors in the Destruction of Road Pavements, by Stephen Taber, Public Roads, vol. 11, No. 6, August, 1930.

moisture in detrimental amounts the pore size must be small enough to furnish appreciable capillary pressure but large enough to prevent too much frictional resistance to flow.[5]

As an indication of the amounts of water furnished by capillarity, it has been estimated that certain Iowa silts (*9*) are capable of raising water by capillarity at the rate of about 5 feet per year. It is indicated by observations in both New Hampshire and Minnesota that in extreme cases the rate may be as high as 10 or 15 feet per year.

Figure 27.—Frozen Cylinder, Half Sand and Half Clay. Much Segregated Ice in Clay but not in Sand. Furnished by Taber Experiments

Generally it can be stated that in cohesionless soils possessing but little capillarity no important frost heaves occurs because practically all of the contained water freezes at normal freezing temperatures and small unfrozen water particles do not exist in amounts sufficient to cause the frozen particles to suffer appreciable growth. Soils possessing relatively high capillarity and relatively low cohesion are likely to heave in very appreciable amounts. The contrast between sand and clay in this respect is shown in Figure 27.[4]

Highly cohesive soils may possess very high capillarity but the resistance to water flow in these soils is very great. Consequently in dense cohesive soils with low ground water level and absence of lateral seepage, only limited amounts of water are available for ice segregation. Under these conditions the soil adjacent to the growing ice crystals, as illustrated in Figure 28, is likely to dry out and shrink because of the loss of moisture (*16*). The ground water elevation in clays, therefore, must be comparatively high in order that important frost heave may occur.

IMPORTANT SUBGRADE CHARACTERISTICS INDICATED BY THE PRESENCE OF CERTAIN SOIL CONSTITUENTS

In the foregoing discussion certain relations have been shown to exist between the five basic physical characteristics of a soil (internal friction, cohesion, compressibility, elasticity, and capillarity) and the important characteristics of subgrade performance, i. e., resistance to lateral flow, the property of compressing vertically under applied loads with or without rebound upon the removal of load, resistance to sliding in cuts and fills, shrinkage or expansion due to changes in moisture, and heaving under frost action.

Both the state in which the soil exists and the properties of the soil constituents exert an important influence upon the occurrence of these characteristics. Thus, for instance, tests performed by Terzaghi (*4*) disclose that the ultimate bearing value (yield point) of a soil in the undisturbed state may be over twice that of the same soil in the disturbed state. Likewise the compressibility may be different when soil is in the disturbed state from what it is in the undisturbed state (*17*).

It is well known that the faces of cuts in certain of the loess soils in the Middle West may stand vertically for years in the undisturbed state whereas these same soils in the disturbed state lose stability easily and flow in the presence of water.

It is also common knowledge that sands may be either highly stable or "quick" depending on whether water, as for instance that furnished by waves, flows downward or, as in the case of that furnished by springs, flows upward through them.

In spite of these facts the importance of tests performed on soils in the disturbed state becomes apparent when one considers (*a*) that the soil composing the subgrade generally, exists in at least a partially manipulated state and that composing the sands-clay and other low type road surfaces exists in a completely manipulated state, and (*b*) that the constants furnished by such tests serve to disclose the presence of those soils constituents which exert an important influence not only on the five basic physical characteristics of subgrades referred to above but also upon the state in which soils may occur in the soil profile. When only the mechanical analysis was used to identify soils, many soils indicated by this determination to be similar in character when disturbed, were observed to be radically different in character when undisturbed in the field. This difference in field behavior was attributed to difference in the environment under which the soil developed or existed. Now it is found that soils similar in character according to the mechanical analysis may differ widely in character according to the physical tests and furthermore that many differences in field behavior of soils are due to differences in soil constituents.

Figure 28.—Type of Shrinkage Cracking Likely to Occur When Water is Drawn Out of Soils Due to Freezing in Adjacent Areas

The subgrade investigations have made it increasingly evident that the states in which soils are likely to exist under specified conditions, as well as the physical characteristics of the soils in different states, may be largely dependent upon the presence of certain soil constituents.

Among the many natural constituents of soils, a comparatively few serve to illustrate the properties which

[4] Reproduced from the article, Freezing and Thawing of Soils as Factors in the Destruction of Road Pavements, by Stephen Tabor, Public Roads, vol. 11, No. 6, August, 1930.

[5] In 1909 Atterberg determined experimentally that the maximum height of capillary rise in 24 hours occurs in soil consisting primarily of particles 0.02 millimeter in diameter. This result was confirmed theoretically by Terzaghi (Erdbaumechanik, 1925, fig. 23).

may exert an important influence upon subgrade performance. These representative soil constituents are:

Gravel.—Particles larger than 2.0 millimeters in diameter (No. 10 sieve).

Coarse sand.—Particles between 0.25 (No. 60 sieve) and 2.0 millimeters in diameter.

Fine sand.—Particles between 0.05 and 0.25 millimeter in diameter.

Silt.—Particles between 0.005 and 0.05 millimeter in diameter.

Cohesive clay.—Particles smaller than 0.005 millimeter in diameter.

Gluey colloids.—According to Albert Atterberg (*18*), soil particles 0.002 millimeter or smaller in diameter show pronounced Brownian movements when suspended in water. This phenomenon, as explained by Oscar Edward Meinzer, indicates that the colloidal stage of fineness is reached at this size and, according to the kinetic theory of heat, the particles are so small that they are bounced about by the rapidly moving molecules with which they collide. The colloidal fraction reported in the mechanical analysis, however, consists of particles 0.001 millimeter and smaller in size.

Mica flakes.

Diatoms.

Peat.

Chemical constituents.—Certain chemicals, such as lime and magnesium, have a tendency to flocculate [6] fine-grained soils; others, such as sodium and potassium have a tendency to deflocculate or disperse fine-grained soils. (Fig. 29.)

Of these constituents, certain of which are shown in the photomicrographs of Figure 30, the gravel and sand are indicative of high internal friction, the silt, peat, and diatoms of detrimental capillarity; the clay and colloidal glues are indicative of cohesion and, together with the silt and when not flocculated, of compressibility; and the mica flakes, peat, and flocculated soils are indicative of elasticity.

Generally gravel and coarse sand furnish the main hardness and supporting strength of graded soil roads, especially in wet weather. Fine sand adds an embedment support to the coarse sand, and silt with low moisture content adds embedment for the sand. Clay and colloidal glues furnish cohesive and adhesive bond variable with their moisture contents (*11*).

The curves of Figure 31 show the great range of compressibility and expansion possessed by different soil constituents.

SUBGRADES TENTATIVELY ARRANGED IN GROUPS

Because of the fact that the presence of certain soil constituents indicates the important soil properties, the subgrades may be arranged in groups representative of both soil constituents and characteristics. On this basis, the various soils have been tentatively arranged in groups with respect to their performance when used as subgrades, as follows (*7*):

UNIFORM SUBGRADES

Group A–1.—Well-graded material, coarse and fine, excellent binder. Highly stable under wheel loads, irrespective of moisture conditions. Functions satisfactorily when surface treated or when used as a base for relatively thin wearing courses.

Group A–2.—Coarse and fine materials, improper grading or inferior binder. Highly stable when fairly dry. Likely to soften at high water content caused either by rains or by capillary rise from saturated lower strata when an impervious cover prevents evaporation from the top layer, or to become loose and dusty in long-continued dry weather.

Group A–3.—Coarse material only, no binder. Lacks stability under wheel loads but is unaffected by moisture conditions. Not likely to heave because of frost nor to shrink or expand in appreciable amount. Furnishes excellent support for flexible pavements of moderate thickness and for relatively thin rigid pavements.

Group A–4.—Silt soils without coarse material, and with no appreciable amount of sticky colloidal clay. Has a tendency to absorb water very readily in quantities sufficient to cause rapid loss of stability even when not manipulated. When dry or damp, presents a firm riding surface which rebounds but very little upon the removal of load. Likely to cause cracking in rigid pavements as a result of frost heaving, and failure in flexible pavements because of low supporting value.

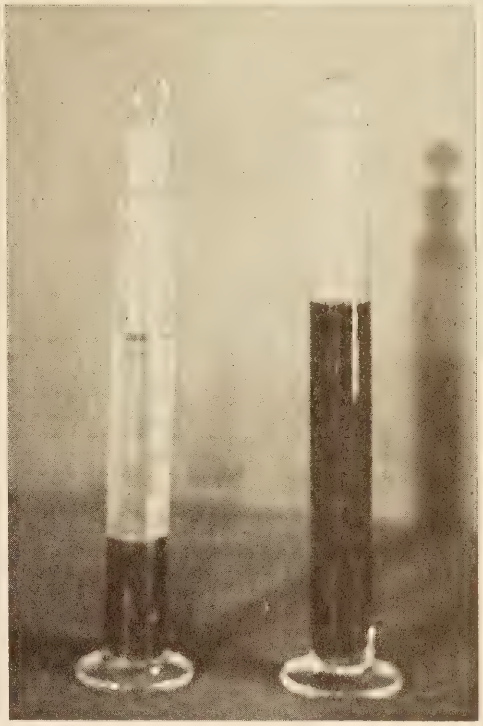


FIGURE 29.—SOIL IN FLOCCULATED AND DISPERSED STATE AFTER 24 HOURS SEDIMENTATION. FLOCCULATED SOIL (LEFT) SETTLES OUT RAPIDLY LEAVING CLEAR LIQUID ABOVE. DISPERSED COLLOIDS REMAIN IN SUSPENSION CAUSING LIQUID TO BE CLOUDY

Group A–5.—Similar to Group A–4, but furnishes highly elastic supporting surfaces with appreciable rebound upon removal of load even when dry. Elastic properties interfere with proper compaction of macadams during construction and with retention of good bond afterwards.

Group A–6.—Clay soils without coarse material. In stiff or soft plastic state absorb additional water only if manipulated. May then change to liquid state and work up into the interstices of macadams or cause failure due to sliding in high fills. Furnish firm support essential in properly compacting macadams only at stiff consistency. Deformations occur slowly and removal of load causes very little rebound. Shrinkage properties combined with alternate wetting and drying under field conditions are likely to cause cracking in rigid pavements.

Group A–7.—Similar to Group A–6, but at certain moisture contents deforms quickly under load and rebounds appreciably upon removal of load, as do subgrades of Group A–5. Alternate wetting and drying under field conditions leads to even more detrimental volume changes than in Group A–6 subgrades. May cause concrete pavements to crack before setting and to crack and fault afterwards. May contain lime or associated chemicals productive of flocculation in soils.

Group A–8.—Very soft peat and muck incapable of supporting a road surface without being previously compacted.

NONUNIFORM SUBGRADES

Soils of these groups cause concrete pavements to crack or fault excessively and flexible types to fail or to develop rough riding surfaces.

Group B–1.—Nonuniform natural ground due to abrupt variation in soil characteristics or soil profile, or to frequent change in field conditions.

Group B–2.—Nonuniform subgrade due to nonuniform composition of fill.

Group B–3.—Nonuniform subgrade consisting in part of natural ground and in part of fill materials.

[6] Arrangement of a number of soil colloids into groups having approximately the size of silt particles.

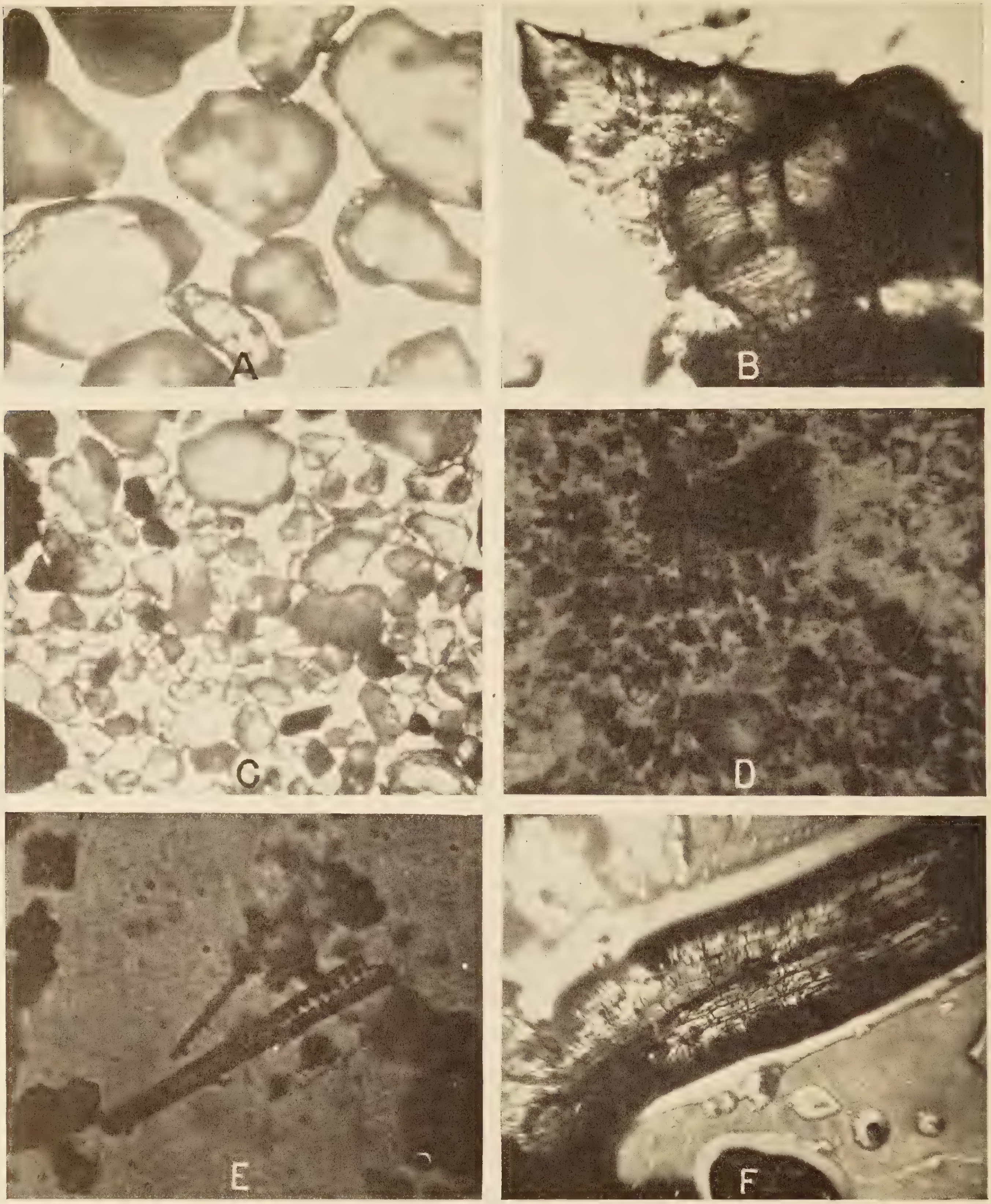


FIGURE 30.—PHOTOMICROGRAPHS OF SOIL CONSTITUENTS. A.—BEACH SAND. B.—ANGULAR SAND GRAIN. C.—GLACIAL SAND. D.—SOIL CONTAINING DIATOMS. NOTE SPONGELIKE APPEARANCE. E.—SINGLE DIATOMS. F.—PEAT-BOG MATERIAL. NOTE FIBROUS STRUCTURE AND FILM OF WATER SURROUNDING INDIVIDUAL PARTICLES

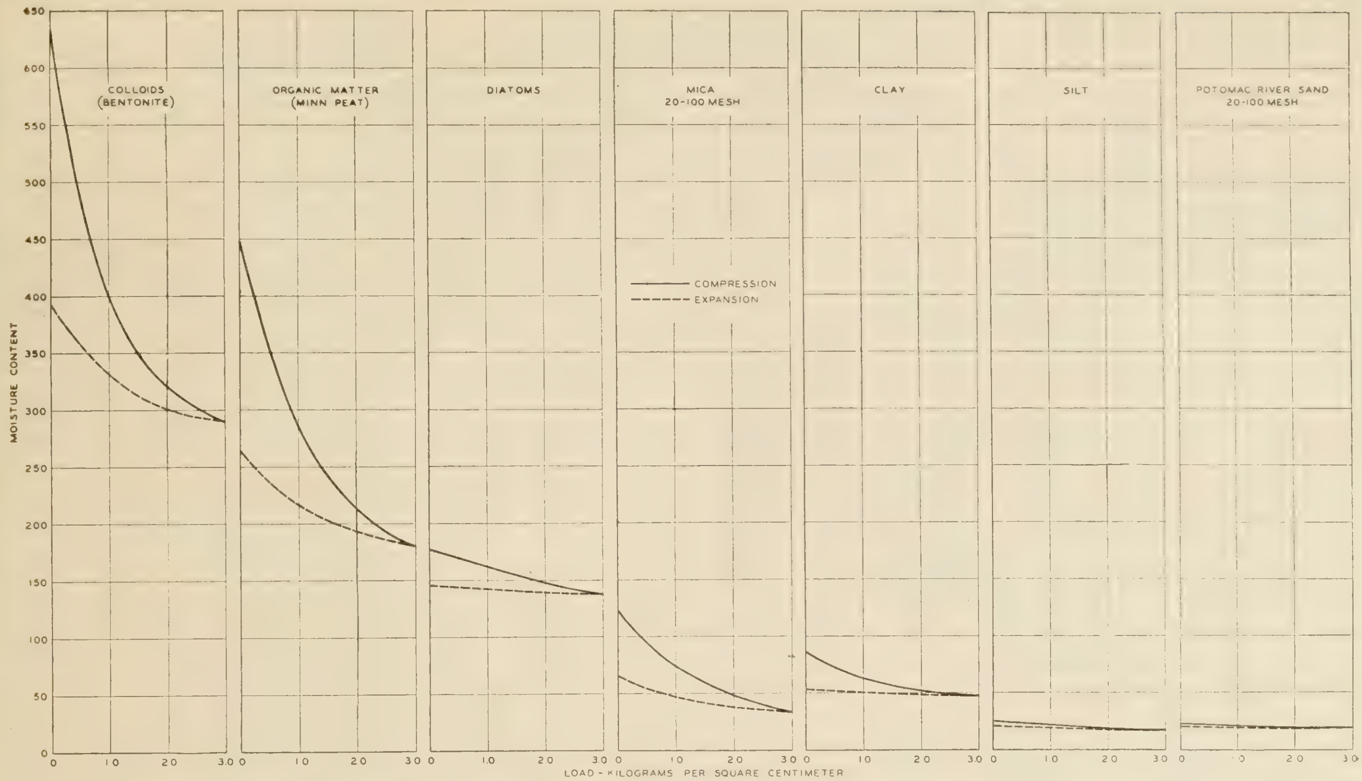


FIGURE 31.—COMPRESSION AND EXPANSION CURVES FOR REPRESENTATIVE SOIL CONSTITUENTS

The foregoing is not intended to be a rigid and final soil classification. It does, however, arrange the uniform soils according to those physical characteristics which are important with respect to subgrade performance and in this manner constitutes the basis for a final classification.

The terms "clay," "silt," and "sand" are used in defining the different groups. These terms, however, refer to the physical properties generally assumed to be possessed by these constituents rather than to definite grain sizes. Thus in the identification of members of the different soil groups grain size is subordinate to physical properties.

The advantage furnished by this method of identification is illustrated as follows: Assume for instance a bulky grained material, Figure 32, A and B, passing the No. 20 and retained on the No. 100 sieve. According to grain size it is sand. Physically it furnishes excellent firm support for a road surface. It does not possess either detrimental elasticity or capillarity and is, consequently, a Group A–3 subgrade.

A similar material containing an appreciable amount of micaceous particles, Figure 32 C, passing the No. 20 and retained on the No. 100 sieve would also be sand, according to grain size. Physically, however, this material possesses both elasticity and capillarity in detrimental amounts; and, since it has no cohesion, it is a Group A–5 subgrade.

In this grouping, therefore, sand instead of being a material characterized only by a specific grain size becomes a Group A–3 subgrade having internal friction, no cohesion, and capillarity in amount insufficient to cause detrimental expansion. Members of the A–3 group are identified with respect to the ability possessed by their grains to resist sliding over each other.

Silt instead of being a material characterized only by a given grain size is divided into two groups: A–4 subgrade, which possesses internal friction, capillarity in appreciable amount, and neither cohesion nor elasticity in appreciable amount; and A–5, subgrade which has internal friction, both capillarity and elasticity in appreciable amount, and no cohesion.

Likewise, the clays are divided into two groups: A–6 subgrade which possesses both cohesion and capillarity in appreciable amount, but neither internal friction nor elasticity; and A–7 subgrade which possesses cohesion, capillarity, and elasticity in appreciable amount, but no internal friction.

This grouping with respect to both the physical properties of the soil and the soil constituents is illustrated diagrammatically in Figure 33. Different positions on the diagram represent different combinations of the five basic physical soil properties. Thus, for instance, a point plotted in the center of the diagram represents the perfect A–3 subgrade, having a maximum amount of internal friction. As the position of the point shifts from the center to the outer boundary of the diagram the magnitude of internal friction decreases gradually from a maximum to negligible amounts. As a point shifts along the circumference from the bottom to the top of the chart the indicated soil changes gradually from the compressible to the elastic type.

Adding compressible materials in increasing amounts therefore gradually changes a group A–3 sand first to a nonplastic variety of the A–2 subgrade; second, to a well graded A–1 subgrade; third, to a plastic variety of the A–2 subgrade and last to either a Group A–4 or Group A–6 subgrade.

The friable variety (sand predominating) differs from the plastic variety (clay predominating) of the Group A–2 subgrade in the following manner. A friable variety, to remain stable, requires the cohesion furnished by capillary pressure and therefore is likely to be highly stable on damp and unstable on thoroughly

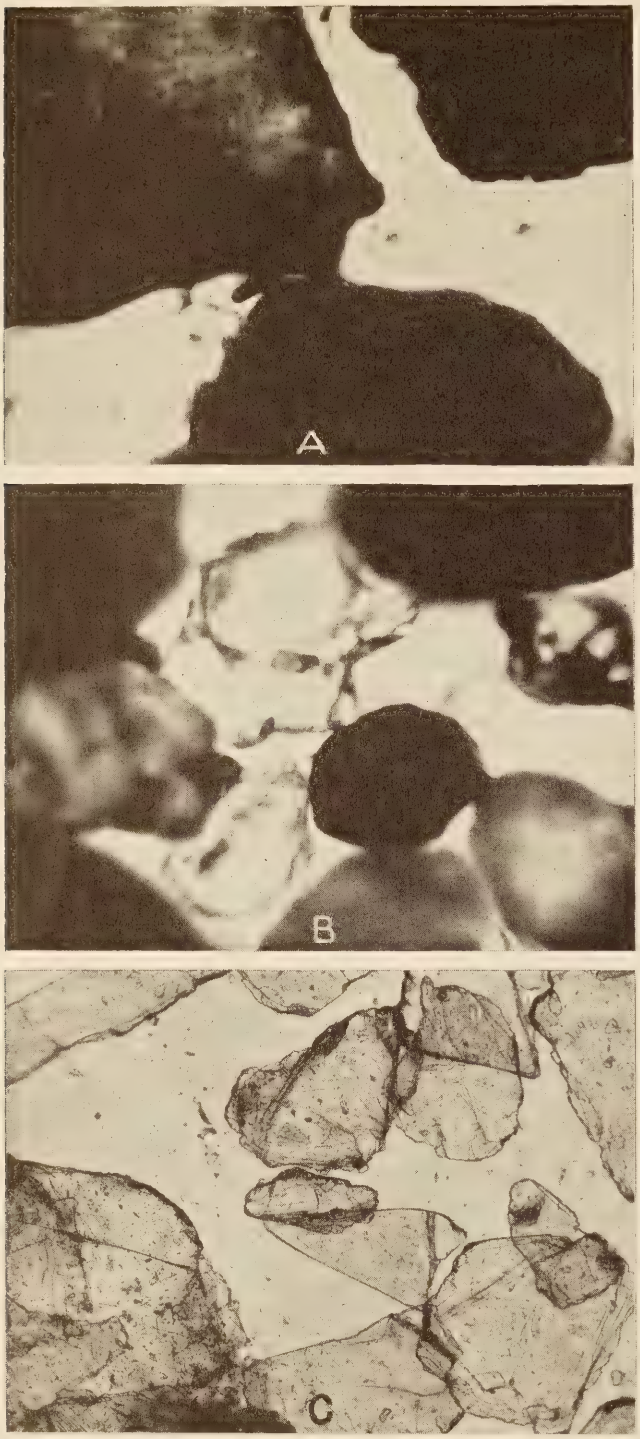


Figure 32.—Photomicrographs of Material Passing the No. 20 and Retained on the No. 100 Sieve. A.—Crushed Angular Sand. B.—Subangular River Sand. C.—Mica Flakes

dry subgrades. The plastic variety, in contrast, remains stable when fairly dry and is apt to soften on damp subgrades.

Thus one sees that the uniform subgrade groups may be defined with respect to properties summarized as follows:

Group A–1.—High internal friction, high cohesion, no detrimental shrinkage, expansion, capillarity, or elasticity.

Group A–2.—High internal friction and high cohesion only under certain conditions. May have detrimental shrinkage, expansion, capillarity, or elasticity.

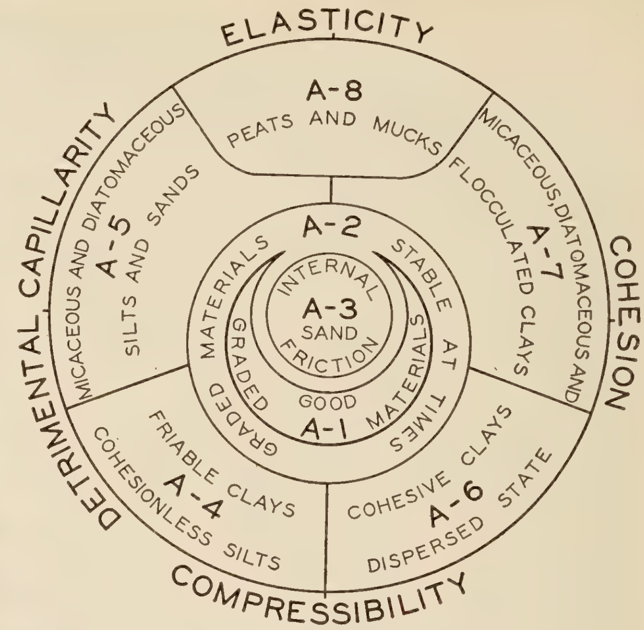


Figure 33.—Diagram Illustrating Characteristics of Uniform Subgrade Groups

Group A–3.—High internal friction, no cohesion, no detrimental capillarity, or elasticity.

Group A–4.—Internal friction variable, no appreciable cohesion, no elasticity, capillarity important.

Group A–5.—Similar to A–4 and in addition possesses elasticity in appreciable amount.

Group A–6.—Low internal friction, cohesion high under low moisture content, no elasticity, likely to expand and shrink in detrimental amount.

Group A–7.—Similar to A–6 but possesses elasticity also.

Group A–8.—Low internal friction, low cohesion, apt to possess capillarity and elasticity in detrimental amount.

BIBLIOGRAPHY

1. Terzaghi, Charles.
   Principles of final soil classification. Public Roads, vol. 8, No. 3, May, 1927.
2. Boyd, J. R.
   Procedure for testing subgrade soils. Public Roads, vol. 6, No. 2, April, 1925.
3. Westergaard, H. M.
   Mechanics of progressive cracking in concrete pavements. Public Roads, vol. 10, No. 4, June, 1929.
4. Terzaghi, Charles.
   Determination of consistency of soils by means of penetration tests. Public Roads, vol. 7, No. 12, February, 1927.
5. Meintzer, O. E.
   The occurrence of ground water in the United States. U. S. Geol. Survey, Water Supply Paper 489, 1928.
6. Hogentogler, C. A., and Aaron, Henry.
   Treatment of highway subgrades with bituminous materials. Eighth Annual Asphalt Paving Conference, 1929.
7. Hogentogler, C. A., and Terzaghi, Charles.
   Interrelationship of load, road, and subgrade. Public Roads, vol. 10, No. 3, May, 1929.
8. Terzaghi, Charles.
   The mechanics of shear failures on clay slopes and the creep of retaining walls. Public Roads, vol. 10, No. 10, December, 1929.
9. Schlick, W. J.
   The theory of underdrainage. Iowa Eng. Exp. Sta. Bulletin 50.
10. Norling, Sven A.
    Subsoil as a factor in drainage. Twenty-first Annual Meeting A. S. A. E., June, 1927.
11. Hogentogler, C. A., and Aaron, Henry.
    Important developments with respect to subgrade investigations. Proc. Ninth Annual Meeting Highway Research Board, 1929.

(Continued on page 116)

THE EFFECT OF WATER-GAS TAR ON THE STRENGTH AND ALKALI RESISTANCE OF CONCRETE

Reported by Dr. E. C. E. LORD, Petrographer, United States Bureau of Public Roads

IN PREVIOUS investigations carried on by the Bureau of Public Roads with tar and paraffin as a means for protecting concrete against alkali attack,[1] no quantitative data were obtained regarding the effect of the tar treatment on the strength of the concrete or its resistance to the action of sulphate solutions under severe service conditions.

It was planned at the outset of the investigation reported herein to carry out strength tests simultaneously on treated and untreated specimens stored out of doors in tap water at Arlington Farm, Va., and in the sulphate water of Medicine Lake, S. Dak. It was found, however, that many of the lake specimens were so severely attacked after a comparatively short period that the relative degree of protection offered by the various tar treatments could best be obtained by recording the conditions of the samples from year to year.

PREPARATION OF SPECIMENS

The concrete test specimens employed in the present investigations were 3 by 6 inch cylinders of 1:2:4 and 1:1½:3 mix, and of medium and wet consistency, made up in batches of 3 each with Potomac River sand (fineness modulus 2.60) and Potomac River gravel graded from three-fourths to one-half inch (60 per cent) and from one-half to one-fourth inch (40 per cent). The cement used passed all standard specification requirements.

The mixes were proportioned with a water-cement ratio of 1.25 for the 1:2:4 mix and 0.86 for the 1:1½:3 mix, of wet consistency (flow 180), and 1.10 for the 1:2:4 mix and 0.83 for the 1:1½:3 mix, of medium consistency (flow 140).

The specimens for compressive strength determinations were cured for 7, 28, and 90 days in damp air and 14 days in dry air for 1:2:4 concrete of medium consistency (Table 1); and 14 days in damp air and 3, 14, 28, and 60 days in dry air for 1:1½:3 and 1:2:4 mixes of medium consistency (Tables 2 and 3). Specimens of 1:1½:3 and 1:2:4 concrete, medium and wet consistency, for exposure to alkali action were cured for 14 days in damp air and 3, 7, 14, 28, and 60 days in dry air (Table 4) before receiving tar treatment.

Immediately after curing, water-gas tar conforming to grade TW-1-X of the provisional specifications for tars for use in the protection of cement concrete[2] was applied to the concrete in brush coats. The quantity of tar absorbed after each application was determined separately and calculated on a percentage basis from the weight of the untreated specimen. Following the priming coats of water-gas tar alternate batches of 3 cylinders each were given one seal coat of coal tar conforming to grade TR-1-25 of the provisional specifications.

After treatment, the samples for alkali exposure were shipped in specially prepared crates, each containing 8 batches (24 specimens), to Medicine Lake, S. Dak.[3]

The specimens for strength determination were treated in the same way and stored at Arlington Farm, Va., in a galvanized iron tank of about 400 gallons capacity, in which the water was frequently renewed.

TABLE 1.—*Compressive strength of 1:2:4 concrete of medium consistency treated with water-gas tar and coal tar after storage from one week to one year in tap water* [a]

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
Batch No.	Curing	Treatment	Tar absorbed	Penetration	Strength after stated number of weeks in water (pounds per square inch)						Per cent increase in strength after stated numberof weeks [b]					Strength rating after stated number of weeks in water				
					1	2	4	13	26	52	2	4	13	26	52	2	4	13	26	52
	Days	*No. of coats*	*P. ct.*	*Inch*												*P. ct.*	*P. ct.*	*P. ct.*	*P.ct.*	*P.ct.*
1	7 damp air, 14 dry air.	Untreated	------	------	1,968	1,998	2,201	2,630	2,588	3,053	1.5	11.8	33.7	31.5	55.1	100	100	100	100	100
2	do	4 water-gas tar	0.42	⅛	2,065	1,971	2,014	2,503	2,360	2,775	-4.6	-2.5	21.2	14.3	34.4	98.6	91.5	95.2	91.2	90.9
3	do	4 water-gas tar, 1 coal tar.	0.77	⅛	1,645	1,905	2,039	2,282	2,227	2,675	15.8	24.0	38.7	35.4	62.7	95.3	92.6	86.8	86.1	87.6
4	do	8 water-gas tar	0.93	¼	1,672	2,014	2,144	2,033	2,183	2,679	20.5	28.2	21.6	30.6	60.2	100.8	97.4	77.3	84.4	87.7
5	do	8 water-gas tar, 1 coal tar.	1.36	¼	1,681	2,018	1,963	2,157	1,983	2,386	20.0	16.8	28.3	18.0	41.9	101.0	89.2	82.0	76.6	78.1
6	28 damp air, 14 dry air.	Untreated	------	------	------	------	------	------	3,637	3,655	------	------	------	------	0.5	------	------	------	100	100
7	do	4 water-gas tar	0.20	1/16	------	------	------	------	3,476	2,895	------	------	------	------	-16.7	------	------	------	95.5	79.2
8	do	4 water-gas tar, 1 coal tar.	0.78	1/16	------	------	------	------	2,885	2,935	------	------	------	------	1.7	------	------	------	79.3	80.3
9	do	6 water-gas tar	0.62	⅛	------	------	------	------	3,228	3,010	------	------	------	------	-6.7	------	------	------	88.7	82.3
10	do	6 water-gas tar, 1 coal tar.	0.93	⅛	------	------	------	------	2,882	2,970	------	------	------	------	3.1	------	------	------	79.2	81.3
11	90 damp air, 14 dry air.	Untreated	------	------	------	------	------	------	3,395	2,923	------	------	------	------	-13.9	------	------	------	100	------
12	do	4 water-gas tar	0.37	1/32	------	------	------	------	2,653	2,793	------	------	------	------	5.3	------	------	------	78.2	------
13	do	4 water-gas tar, 1 coal tar.	0.69	1/32	------	------	------	------	2,792	2,795	------	------	------	------	0.1	------	------	------	82.2	------
14	do	do	0.62	1/32	------	------	------	------	2,765	2,936	------	------	------	------	6.2	------	------	------	81.5	------

[a] Test results indicate averages of three samples in each batch.
[b] Per cent increase in strength was calculated from values obtained at one week and six months.

[1] Public Roads, vol. 5, No. 3, May, 1924; vol. 6, No. 11, January, 1926; and vol. 8, No. 6, August, 1927.
[2] Public Roads, vol. 8, No. 6, August, 1927, p. 107.
[3] Medicine Lake is situated 18 miles northwest of Watertown, S. Dak., and is extremely rich in alkali, the sulphate content of the water varying according to the season of the year from 2.14 to 3.06 per cent magnesium sulphate and from 0.12 to 1.34 per cent sodium sulphate. Public Roads, vol. 6, No. 8, October, 1925, p. 175.

TABLE 2.—*Compressive strength of 1:1½:3 concrete, medium consistency, treated with water-gas tar and coal tar after storage from 30 to 40 months in tap water*

1	2	3	4	5	6	7	8	9	10	11	12	13
Batch No.	Crate No.	Samples tested	Curing	Treatment	Tar absorbed	Penetration	Strength after stated number of months in water, pounds per square inch				Strength deviation[1]	Strength rating
							30	34	38	40		
			Days	*No. of coats*	*P. ct.*	*Inch*					*Per cent*	*Per cent*
1	110	3	14 damp air, 3 dry air	Untreated						4,460	8.1	
2		3	do	do						4,773	1.6	
3		3	do	do						5,157	6.3	
4		3	do	do						4,927	1.5	
5		2	do	do						4,985	2.7	
Crate total and averages		14								4,852		100
6	90	3	14 damp air, 3 dry air	2 water-gas tar	0.70	1/32				4,823	−0.7	99.4
7		3	do	2 water-gas tar, 1 coal tar	1.17	1/32				4,837	−0.4	99.7
8		3	do	3 water-gas tar	.88	1/32				4,947	+1.9	101.9
9		3	do	3 water-gas tar, 1 coal tar	1.44	1/32				5,370	+10.6	110.7
10		3	do	4 water-gas tar	1.15	1/16				4,263	−12.2	87.8
11		3	do	4 water-gas tar, 1 coal tar	2.04	1/16				5,033	+3.6	103.7
12		3	do	5 water-gas tar	1.23	1/16				5,080	+4.6	104.8
13		3	do	5 water-gas tar, 1 coal tar	1.89	1/16				4 503	−7.3	92.7
Crate total and averages		24		4 coats	1.31	1/21				4,857		100.1
14	98	3	14 damp air, 14 dry air	4 water-gas tar	.62	1/16			3,930		+0.6	
15		3	do	4 water-gas tar, 1 coal tar	.90	1/16			3,963		+1.5	
16		3	do	6 water-gas tar	.76	1/8			4,070		+4.2	
17		3	do	6 water-gas tar, 1 coal tar	1.08	1/8			4,150		+6.3	
18		3	do	6 water-gas tar	.82	1/8			3,863		−1.1	
19		3	do	6 water-gas tar, 1 coal tar	1.11	1/8			4,157		+6.5	
20		3	do	6 water-gas tar	1.06	1/8			3,673		−5.9	
21		3	do	6 water-gas tar, 1 coal tar	.94	1/8			3,437		−12.0	
Crate total and averages		24		6 coats	.91	1/9			3,905			
22	102	3	14 damp air, 28 dry air	Untreated				4,660				100
23		2	do	3 water-gas tar, 1 coal tar	.96	1/16		4,650			−0.9	99.8
24		3	do	4 water-gas tar	.69	1/16		4,617			−1.6	99.1
25		3	do	4 water-gas tar, 1 coal tar	.91	1/16		4,647			−1.0	99.7
26		3	do	5 water-gas tar	.69	1/8		4,883			+4.0	104.8
27		3	do	5 water-gas tar, 1 coal tar	1.06	1/8		4,663			−0.7	100.1
28		3	do	5 water-gas tar	.65	1/8		4,690			−0.1	100.7
29		3	do	5 water-gas tar, 1 coal tar	1.04	1/8		4,697			+0.1	100.8
Crate total and averages		23		5 coats	.86	1/10		[2] 4,694				100.6
30	106	3	14 damp air, 60 dry air	Untreated			5,163					100
31		3	do	2 water-gas tar, 1 coal tar	.69	1/64	4,820				−5.2	93.3
32		3	do	3 water-gas tar	.38	1/64	5,140				+1.1	99.5
33		3	do	3 water-gas tar, 1 coal tar	.74	1/64	5,163				+1.5	100.0
34		3	do	4 water-gas tar	.47	1/32	5,277				+3.8	102.2
35		3	do	4 water-gas tar, 1 coal tar	.47	1/32	5,140				+1.1	99.5
36		3	do	4 water-gas tar	.53	1/32	4,980				−2.1	96.4
37		3	do	4 water-gas tar, 1 coal tar	.80	1/32	5,080				−0.1	98.4
Crate total and averages		24		4 coats	.58	1/41	[2] 5,086					98.7

[1] Strength deviation indicates averages of each batch compared with the crate average.
[2] Average of treated samples.

TREATED AND UNTREATED CYLINDERS SHOW LITTLE DIFFERENCE IN ULTIMATE STRENGTH AFTER STORAGE IN TAP WATER

In Table 1 are given the results of compression tests of 1:2:4 concrete cylinders of medium consistency, treated in batches of 3 cylinders each with water-gas tar and coal tar after storage from one week to one year in tap water.

It will be observed that tar absorption (columns 4 and 5) decreased as time of curing increased (column 2), and that the strength developed is at a minimum with maximum tar absorption for samples cured for a short period of time (batches 2 to 5, columns 11 and 21). The results indicate that the average increase in strength for the treated material between the ages of 1 week and 1 year is about the same as for the untreated material (batches 1 to 5, columns 12 to 16), but the strength of the treated samples after one year in water averages about 14 per cent less than that of the untreated specimens (column 21).

For samples cured 28 days in damp air and 14 days in dry air before receiving the tar treatment (batches 6 to 10) the results were inconclusive, some batches indicating a slight gain (batches 8 and 10, column 16) and others an appreciable loss in strength during the six months to one year period (batches 7 and 9, column 16). The average strength rating of these samples after one year was about 81 (column 21).

A more consistent gain in strength was indicated by specimens cured for 90 days in damp air and 14 days in dry air (batches 12 to 14, column 16) where the values obtained indicated an average gain of about 4 per cent between 6 months and 1 year in water. A retrogression in strength of over 16 per cent between 6 months and 1 year for the untreated specimens belonging to this group (batch 11, column 16) would render the computation of the strength ratings at the age of 1 year misleading, but if computed on the basis of the strength of the untreated samples after 26 weeks in water (batch 11, column 10) the strength ratings of these batches would average essentially the same at the age of 6 months as those of the foregoing group at the age of 1 year, or about 81.

From these results it may be concluded that the tar treatment retards strength development in concrete up to a period of 1 year in water; the loss in strength averaging from about 15 to 20 per cent compared with that of the untreated material.

It will be observed from the following data, however, that after longer storage the effect of the tar treatment on the strength of concrete is much less apparent.

TABLE 3.—*Compression strength of 1:2:4 concrete, medium consistency, treated with water-gas tar and coal tar after storage from 30 to 40 months in tap water*

1	2	3	4	5	6	7	8	9	10	11	12	13
Batch No.	Crate No.	Samples tested	Curing	Treatment	Tar absorbed	Penetration	Strength after stated number of months in water, pounds per square inch				Strength deviation[1]	Strength rating
							30	34	38	40		
			Days	*No. of Coats*	*P. ct.*	*Inch*					*Per cent*	*Per cent*
1	92	3	14 damp air, 3 dry air	Untreated						3,740		100
2		3	do	2 water-gas tar, 1 coal tar	1.05	1/16				3,740	+6.6	100.0
3		3	do	3 water-gas tar	1.02	1/16				3,630	+3.5	97.1
4		3	do	3 water-gas tar, 1 coal tar	1.23	1/16				3,443	−1.8	92.0
5		3	do	4 water-gas tar	1.00	1/8				3,663	+4.4	98.0
6		3	do	4 water-gas tar, 1 coal tar	1.46	1/8				3,227	−8.0	86.3
7		3	do	5 water-gas tar	1.16	1/8				3,170	−9.6	84.8
8		3	do	5 water-gas tar, 1 coal tar	1.49	1/8				3,677	+4.8	98.3
Crate total and averages		24		4 coats	1.20	1/10				[2] 3,507		94.6
9	100	3	14 damp air, 14 dry air	3 water-gas tar	.64	1/8		2,977			−3.3	
10		3	do	3 water-gas tar, 1 coal tar	1.02	1/8		2,860			−7.1	
11		3	do	4 water-gas tar	.82	1/8		3,083			+0.1	
12		3	do	4 water-gas iar, 1 coal tar	1.05	1/8		3,420			+11.1	
13		3	do	5 water-gas tar	.88	1/8		3,093			+0.5	
14		3	do	5 water-gas tar, 1 coal tar	1.14	1/8		3,123			+1.4	
15		3	do	6 water-gas tar	.94	1/4		3,200			+3.9	
16		3	do	6 water-gas tar, 1 coal tar	1,15	1/4		2,873			−6.7	
Crate total and averages		24		5 coats	0.96	1/6		3,079				
17	104	3	14 damp air, 28 dry air	Untreated				3,260				100
18		3	do	3 water-gas tar, 1 caol tar	1.02	1/16		2,980			−2.9	91.4
19		3	do	4 water-gas tar	.82	1/16		2,923			−4.8	89.7
20		3	do	4 water-gas tar, 1 coal tar	1.13	1/8		3,057			−0.4	93.8
21		3	do	5 water-gas tar	.82	1/8		3,030			−1.3	93.0
22		3	do	5 water-gas tar, 1 coal tar	1,10	1/8		3,223			+5.0	98.9
23		3	do	6 water-gas tar	.91	1/4		3,107			+1.2	95.3
24		3	do	6 water-gas tar, 1 coal tar	1.35	1/4		3,170			+3.3	97.2
Crate total and averages		24		5 coats	1.02	1/7		[2] 3,070				95.0
25	108	3	14 damp air, 60 dry air	Untreated			3,503					100
26		3	do	2 water-gas tar, 1 coal tar	.88	1/8	3,637				+1.6	103.8
27		3	do	3 water-gas tar	.62	1/8	3,693				+3.2	105.4
28		3	do	3 water-gas tar, 1 coal tar	1.08	1/8	3,267				−8.7	93.2
29		3	do	4 water-gas tar	.85	1/4	3,413				−4.7	97.4
30		3	do	4 water-gas tar, 1 coal tar	1.20	1/4	3,440				−3.9	98.2
31		3	do	5 water-gas tar	.94	1/4	3,603				+0.6	102.8
32		3	do	5 water-gas tar, 1 coal tar	1,14	1/4	4,010				+12.0	114.5
Crate and total averages		24		4 coats	0.96	1/6	[2] 3,580					101.9

[1] Strength deviation indicate averages of each batch compared with the crate average.
[2] Average of treated samples.

In Tables 2 and 3 are recorded the results of compression tests on 1:1½:3 and 1:2:4 concrete of medium consistency after 30 to 40 months in tap water. The specimens are duplicates of those receiving the same treatment (column 5) that were sent to Medicine Lake for exposure to alkali action and have been given the same crate numbers (column 2).

The test results indicating averages of samples from each batch, as well as averages of all samples from each crate are given in columns 6 to 13. The average strength of the individual batches of 3 cylinders each receiving varying quantities of tar was found to deviate somewhat from that of the crate as a whole and this deviation, computed in percentage (column 12) has served as an index for comparison where strength ratings were not obtained.

It will be observed that the average quantity of tar absorbed reached a maximum in samples of both mixes cured for 14 days in damp air and 3 days in dry air (Table 2, batches 6 to 13, column 6, and Table 3, batches 2 to 8, column 6), and that in the richer mixes the quantity absorbed decreases with increase in time of curing until a minimum is reached at 60 days in dry air (Table 2, batches 31–37). With the leaner mixes the amount absorbed is nearly uniform beyond a curing period of 3 days in dry air (Table 3, batches 9–32). The extent of penetration varies considerably, being appreciably greater in the leaner mixes (Table 3, column 7).

These results indicate that the constituents of the hydrated cement are in an extremely tar-absorbent condition shortly after removal of the concrete from damp-air curing; but that after exposure to dry air these compounds, essentially colloidal in character, presumably undergo a change in structure and become less absorptive through loss of moisture and the action of atmospheric agencies.

Regarding the effect of the tar treatment on the strength of concrete after 2 years and more in water, the test results indicate in the case of the richer mixes (Table 2) that while a slight reduction in strength has occurred in some batches the average of the crate groups about equals that of the untreated material (Table 2, columns 8 to 13).

As regards the leaner mixes (Table 3), the results indicate that while certain batches have fallen off in strength, especially for samples cured for shorter periods in dry air (batches 6 and 7, columns 11 to 13), the average strength for each crate approaches closely to that of the untreated material (Table 3, columns 8 to 13).

From an examination of Tables 2 and 3 it is evident that no definite relationship can be found between quantity of tar absorbed and strength development. Many batches receiving a maximum quantity of tar developed a strength as great as or greater than those receiving a minimum quantity (batches 6 and 11, 16, and 19, 32 and 37, in Table 2, and batches 3 and 8, 9 and 14, 19 and 24, in Table 3).

TABLE 4.—*Exposure tests of concrete treated with water-gas tar and coal tar and stored in alkali water at Medicine Lake, S. Dak.*

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
Batch No.	Crate No.	Mix	Consistency	Curing	Treatment	Average tar absorbed	Samples attacked after stated number of months in lake											Time of failure
							6	9	12	15	18	21	24	27	33	36	39	
				Days	*No. of Coats*	*Per cent*												*Months*
1	110	1:1½:3	Medium	14 damp air, 3 dry air	Untreated					24				F[1]				27
2	90	1:1½:3	do	do	2, 3, 4, 5, water-gas tar, 1 coal tar	1.34				0				3			3	
3	91	1:1½:3	Wet	do	do	.91				0				2			9	
4	94	1:1½:3	Medium	14 damp air, 7 dry air	4, 6, 8, water-gas tar, 1 coal tar	1.35			1				9			14		
5	95	1:1½:3	Wet	do	4, 6, 7, water-gas tar, 1 coal tar	1.12			0				0			12		
6	98	1:1½:3	Medium	14 damp air, 14 dry air	4, 6, water-gas tar, 1 coal tar	.96		0				2			10			
7	99	1:1½:3	Wet	do	2, 3, 4, 5, water-gas tar, 1 coal tar	.63		6				19			F			33
8	102	1:1½:3	Medium	14 damp air, 28 dry air	3, 4, 5, water-gas tar, 1 coal tar	.86		10				22			F			33
9	103	1:1½:3	Wet	do	do	.73		3				19			F			33
10	106	1:1½:3	Medium	14 damp air, 60 dry air	2, 3, 4, water-gas tar, 1 coal tar	.58	4				16				F			33
11	107	1:1½:3	Wet	do	2, 3, 4, 5, water-gas tar, 1 coal tar	.77	24				F							18
12	111	1:1½:3	Medium	14 damp air, 7 dry air	2 water-gas tar, paraffin, and linseed oil.	.63			0				3					
13	112	1:1½:3	do	do	3, 4, water-gas tar, paraffin, and linseed oil.	.72			0				0					
14	92	1:2:4	do	14 damp air, 3 dry air	2, 3, 4, 5, water-gas tar, 1 coal tar	1.01				8				12			12	
15	93	1:2:4	Wet	do	do	.91				19				F				27
16	96	1:2:4	Medium	14 damp air, 7 dry air	4, 6, 8, water-gas tar, 1 coal tar	1.40			1				12			24		
17	97	1:2:4	Wet	do	do	1.35			6				21			F		36
18	100	1:2:4	Medium	14 damp air, 14 dry air	3, 4, 5, 6, water-gas tar, 1 coal tar	.98		24				F						21
19	101	1:2:4	Wet	do	4, 6, 5, 7, water-gas tar, 1 coal tar	1.01		24				F						21
20	104	1:2:4	Medium	14 damp air, 28 dry air	3, 4, 5, 6, water-gas tar, 1 coal tar	.99	24				F							18
21	105	1:2:4	Wet	do	do	.86	24				F							18
22	108	1:2:4	Medium	14 damp air, 60 dry air	2, 3, 4, 5, water-gas tar, 1 coal tar	.94	10				F							18
23	109	1:2:4	Wet	do	do	.93	12				F							18
24	113	1:2:4	Medium	14 damp air, 7 dry air	2 water-gas tar and paraffin	.74		21				F						21
25	114	1:2:4	do	do	3, 4, water-gas tar and paraffin	.98		24				F						21
26	115	1:2:4	do	do	2, 3, 4, water-gas tar and lubricating oil.	.97		0				F						21
27	116	1:2:4	do	do	2, 4, water-gas tar and paraffin	.77		0				F						21
28	117	1:2:4	do	do	2 water-gas tar and paraffin	.80		0				F						21
29	118	1:2:4	do	do	4, 5, water-gas tar and paraffin	1.38	0				F							18

[1] F indicates failure of all specimens.

From the results thus far obtained it may be concluded that while the tar treatment somewhat retards the early strength development of concrete stored in water, the effect of the treatment decreases with time of storage, the average strength ultimately attained varying but little from that of the untreated material.

RESULTS OF EXPOSURE TESTS INDICATE VALUE OF WATER-GAS TAR TREATMENT UNDER PROPER CONDITIONS OF MIX, CONSISTENCY, AND CURING

The cylinders placed in Medicine Lake for exposure to alakli action were prepared and treated in the same way as those used in the strength tests except that mixes of both medium and wet consistency were employed, and in some cases (Tables 4, batches 12, 13, and 24 to 29) a small quantity (10 per cent by weight of tar) of paraffin, linseed oil or lubricating oil (viscosity 2.85, Saybolt) was incorporated with the tar.

The test results are given in Table 4, in which the indicated values represent averages of 24 specimens tested in each crate.

It may be noted that the indicated quantity of tar absorbed is of the same general order as that previously recorded for the strength tests. That is to say, samples of both mixes cured for shorter periods in dry air (batches 2 to 5 and 14 to 17, columns 5 to 7) are, in general, more absorptive than those cured for longer periods (batches 6 to 11 and 18 to 23, columns 5 to 7). The results indicate also that the dryer mixes are, on the whole, more tar absorptive than the wet mixes cured for the same length of time (batches 2 to 11, 14 to 23, columns 4 and 7).

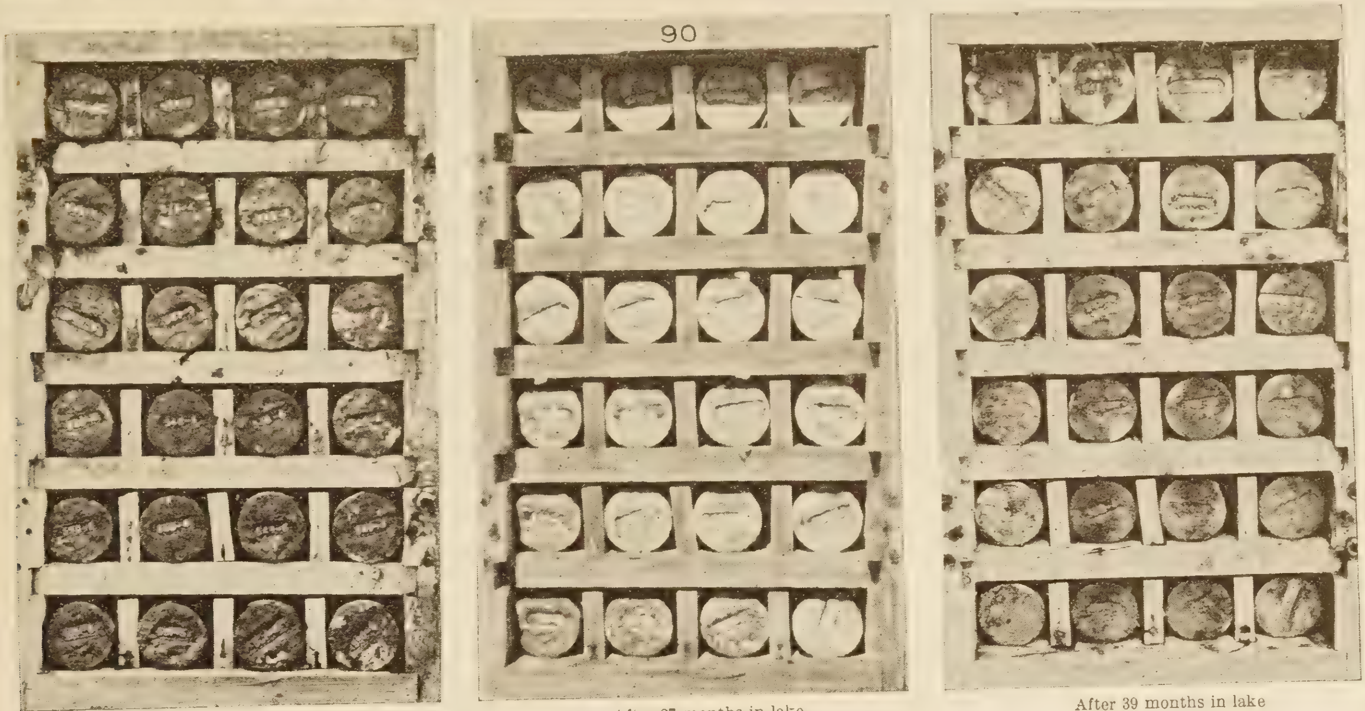
The effect of alkali action on the specimens is indicated in columns 8 to 19, in which the number of specimens attacked in each crate is given, as of the time of inspection.

It will be observed that after an exposure period of from 6 to 15 months (columns 8 to 11) all samples of untreated 1:1½:3 concrete, medium consistency, cured for 14 days in damp air and 3 days in dry air had been attacked (batch. 1, column 11) while samples of the same mix, medium and wet consistency, cured in the same way and receiving from 2 to 5 coats of water-gas tar and 1 coat of coal tar were intact (batches 2 and 3, column 11). It will be noted also that while the treated samples of the same mix cured for 7 days in dry air were in a satisfactory condition after 12 months in the lake (batches 4 and 5, column 10), the protective effect of the tar treatment decreased in efficiency with prolonged curing in dry air until with 60 days' curing all samples of wet consistency were attacked after 6 months exposure (batch 11, column 8).

In the case of the leaner mixes (batches 14 to 23) the tar treatment was not so effective, but it will be noticed that samples cured for a short period in dry air before treatment, especially those of medium consistency, were in better condition after 12 to 14 months exposure (batches 14 to 17, columns 10 and 11) than those cured for a greater length of time (batches 18 to 23, columns 8 and 9).

In regard to the relative protection offered by the tar treatment on concrete of medium and wet consistency (columns 4 and 7 to 19) the test results indicate that, in general, the dryer mixes are more tar absorptive and offer greater resistance to alkali attack.

In the tabulation of the final results of the tests (column 19) it will be observed that all batches of 1:2:4 concrete receiving the tar treatment had failed within a period of from 18 to 36 months in the lake water, except those of medium consistency cured for 14 days in damp air and from 3 to 7 days in dry air (batches 14 and 16, column 19). In the case of the 1:1½:3 specimens only batches cured for 14, 28, and 60 days in dry air had failed within this period (batches 7 to 11, column 19), the remaining batches, especially the batch of medium consistency, cured for 14 days in damp air and 3 days in dry air (batch 2, column 18) being



After 15 months in lake

After 27 months in lake

After 39 months in lake

Figure 1.—Condition of 1:1½:3 Concrete Cylinders of Medium Consistency (Batch 2, Crate 90) After 15, 27, and 39 Months in Medicine Lake, S. Dak. Curing: 14 Days in Damp Air, 3 Days in Dry Air. Treatment: 2, 3, 4, and 5 Coats of Water-Gas Tar and 1 Coat of Coal Tar

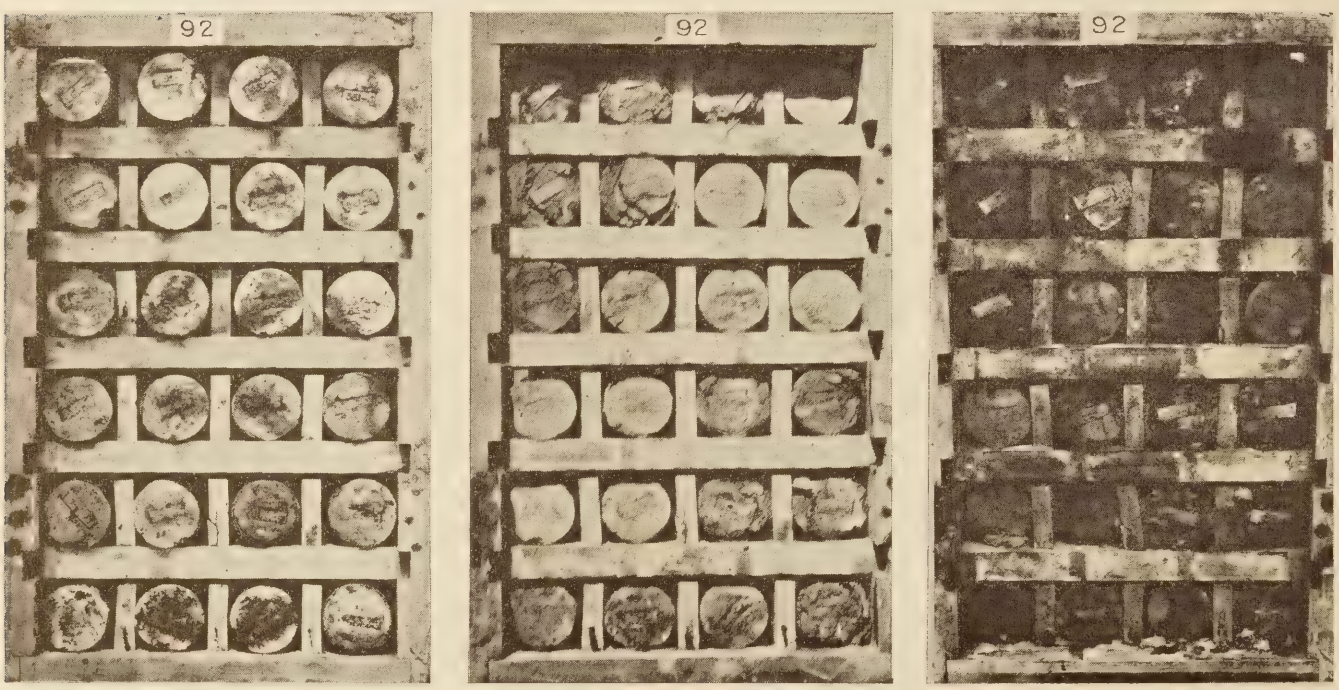
After 15 months in lake

After 27 months in lake

Figure 2.—Condition of Untreated 1:1½:3 Concrete Cylinders of Medium Consistency (Batch 1, Crate 110) After 15 and 27 Months in Medicine Lake, S. Dak. Curing Same as in Figure 1

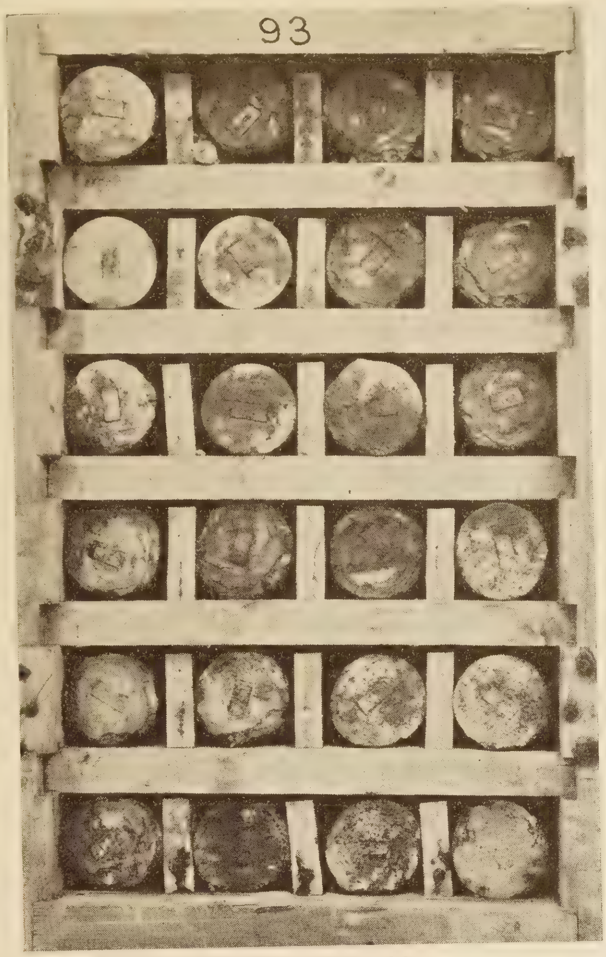
After 15 months in lake | After 27 months in lake | After 39 months in lake

Figure 3.—Condition of 1:1½:3 Concrete Cylinders of Wet Consistency (Batch 3, Crate 91) After 15, 27, and 39 Months in Medicine Lake, S. Dak. Curing and Treatment Same as in Figure 1



After 15 months in lake | After 27 months in lake | After 39 months in lake

Figure 4.—Condition of 1:2:4 Concrete Cylinders of Medium Consistency (Batch 14, Crate 92) After 15, 27, and 39 Months in Medicine Lake, S. Dak. Curing and Treatment Same as in Figure 1



After 15 months in lake

FIGURE 5.—TEST SHOWING CONDITION OF 1:2:4 CONCRETE CYLINDERS OF WET CONSISTENCY (BATCH 15, CRATE 93) AFTER 15 MONTHS IN MEDICINE LAKE, S. DAK. CURING AND TREATMENT SAME AS IN FIGURE 1. THESE SPECIMENS FAILED AFTER 27 MONTHS IN THE LAKE

in good condition. The condition of the cylinders in this batch, after 15, 27, and 39 months exposure, is shown in Figure 1.

Regarding the addition of paraffin and lubricating oil to the tar, the results indicate that these mixtures are ineffectual in retarding alkali attack, samples thus treated having failed within 18 to 21 months in the lake (batches 24 to 29, column 19). The addition of linseed oil, however, to the tar and paraffin mixture appears to be beneficial when applied to the richer mixes, as the samples receiving this treatment (batches 12 and 13, column 14) were practically unattacked after 24 months in the lake (see fig. 6). Linseed oil was not used with the leaner mixes.

PHOTOGRAPHS SHOW CONDITION OF SAMPLES

The condition of the samples after exposure to alkali action may be seen from photographs of the crates taken immediately after their removal from the lake at the time the yearly inspections were made. It should be stated that the samples receiving the minimum quantity of tar were placed in the upper tier in the crates and those receiving progressively larger quantities in the ones below.

The condition of untreated 1:1½:3 concrete of the same consistency and curing as the treated samples in crate No. 90 (fig. 1), after 15 and 27 months in the lake is shown in Figure 2.

It will be noticed that where alkali action had progressed to an appreciable extent the specimens attacked were about evenly distributed throughout the crates (fig. 3, right, fig. 4, center and right, and fig. 5, batches 3, 14, and 15, crates 91, 92, and 93) indicating that the efficiency of the treatment can not be estimated with certainty by the quantity of tar absorbed nor by the addition of a coal tar seal coat. It will be seen later on that the degree of protection is influenced to a greater extent by richness and consistency of mix and method of curing the concrete previous to the treatment than by the quantity of tar applied.

This is brought out by comparing the 1:1½:3 cylinders of medium consistency cured for 14 days in damp air and 3 days in dry air (fig. 1, batch 2, crate 90) with those of wet consistency cured in the same manner (fig. 3, batch 3, crate 91). In crate 90 all the cylinders were in good condition after 39 months in the lake except 3 in the upper tier which had received only 2 coats of water-gas tar (12 grams per cylinder), whereas in crate 91 (fig. 3, right) at least 9 cylinders which had received as much as 16 grams of water-gas tar and 5 grams of coal tar each were badly attacked.

The protective effect of the tar treatment on 1:1½:3 concrete decreases with time of curing in dry air, and it has been shown that all samples cured for longer than 14 days in dry air before treatment failed after 18 to 33 months in the lake water. (Table 4, batches 8 to 11, column 19.)

Batch 12, Crate 111. Batch 13, Crate 112

Curing: 14 days in damp air, 7 days in dry air
Treatment: Batch 12, 2 coats of water-gas tar containing 10 per cent paraffin and 10 per cent linseed oil; batch 13, 3, and 4 coats of water-gas tar containing 10 per cent paraffin and 10 per cent linseed oil

FIGURE 6.—CONDITION OF 1:1½:3 CONCRETE CYLINDERS OF MEDIUM CONSISTENCY, TREATED WITH WATER-GAS TAR CONTAINING PARAFFIN AND LINSEED OIL, AFTER 24 MONTHS IN MEDICINE LAKE, S. DAK.

In the case of the 1:2:4 concrete the conditions are similar to those of the richer mixes although the effect of alkali action is much more evident. This fact may be observed by comparing specimens of medium consistency cured 14 days in damp air and 3 days in dry air (fig. 4, batch 14, crate 92) with samples of the same mix and curing but of wet consistency (fig. 5 batch 15, crate 93). While the latter samples were seriously attacked after 15 months' exposure and had failed completely within 27 months, about one-half the samples of medium consistency were intact at the end of this period (fig. 4, center). These same samples were still in good condition at the end of 39 months (fig. 4, right). As indicated in the table, all specimens of 1:2:4 concrete cured for more than 7 days in dry air had failed within 18 and 21 months' exposure to the lake water. (Table 4, batches 18 to 23, column 19.)

A possible benefit derived by the addition of paraffin and linseed oil to the water-gas tar when applied to 1:1½:3 concrete is indicated in Figure 6 (crates 111 and 112, batches 12 and 13). It should be recalled, however, that all samples of 1:2:4 concrete treated with paraffin and lubricating oil had failed after 22 months' exposure (Table 4, batch Nos. 24 to 29, column 19), and it seems doubtful if the addition of linseed oil to the tar would be of great permanent benefit.

CONCLUSIONS SUMMARIZED

The results of the present investigations lead to the following conclusions:

1. The compressive strength of concrete cured for varying periods in damp and dry air and treated with 4 or more coats of water-gas tar and 1 coat of coal tar is somewhat lowered during early stages of hardening under water, but this loss is ultimately almost fully regained.
2. During the early stages the strength decreases rather uniformly with increasing quantities of tar absorbed, but when ultimate strength is attained the effect of the tar is negligible.
3. The protection against alkali action afforded concrete by the tar treatment is influenced by the cement content, method of curing and consistency of mix, being most effective in rich mixes of medium consistency cured for a minimum length of time in dry air.
4. Concrete of wet consistency is less tar-absorptive and offers lower resistance to alkali attack than concrete of medium consistency cured in the same manner, although no positive relation is shown to exist between the amount of tar absorbed and the resistance to alkali attack.
5. The protection to concrete afforded by the water-gas tar treatment, under the severe conditions imposed in this investigation, was increased only to a limited extent by the application of a nonpenetrable surface coat of coal tar.
6. Concrete of high cement content, cured for a limited period in dry air and treated with water-gas tar is capable of offering appreciable resistance to alkali attack under the conditions imposed in the foregoing investigation.

(Continued from page 108)

12. Terzaghi, Charles.
Simplified soil tests for subgrades and their physical significance. Public Roads, vol. 7, No. 8, October, 1926.
13. Bouyoucos, G. J.
A new classification of soil moisture. Soil Science, vol. 11, No. 1, January to June, 1921.
14. Bouyoucos, G. J.
Movement of soil moisture from small capillaries to the large capillaries of the soil upon freezing. Journal of Agricultural Research, vol. 24, No. 4, 1923.
15. Taber, Stephen.
Frost heave. Journal of Geology, vol. 37, No. 5, July to August, 1929.
16. Mullis, Ira B.
Illustrations of frost and ice phenomena. Public Roads, vol. 11, No. 4, June, 1930.
17. Terzaghi, Charles.
The science of foundations—its present and future. Proc. A. S. C. E., November, 1927.
18. Atterberg, Albert.
Die rational klassifikation der Sande und Kiese. Chem. Zeitung, vol. 29, pt. 1, p. 196, 1905.
19. Ross, C. S., and Shannon, C. V.
The minerals of bentonite and related clays and their physical properties. Journal of American Ceramic Society, vol. 9, February, 1926.
20. Olmstead, L. B., Alexander, L. T., and Middleton, H. C.
A pipette method of mechanical analysis of soils based on improved dispersion procedure. U. S. D. A. Tech. Bul. No. 170, January, 1930.
21. Hogentogler, C. A., Terzaghi, Charles, and Wintermyer, A. M.
Present status of subgrade soil testing. Public Roads, vol. 9, No. 1, March, 1928.
22. Strahan, C. N.
A study of gravel, topsoil, and sand-clay roads in Georgia. Public Roads, vol. 10, No. 7, September, 1929.
23. Hogentogler, C. A.
California road survey demonstrates the economic possibilities of subgrade studies. Public Roads, vol. 7, No. 12, February, 1927.
24. Taber, Stephen.
Freezing and thawing of soils as factors in the destruction of road pavements. Public Roads, vol. 11, No. 6, August, 1930.

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Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.
Report of the Chief of the Bureau of Public Roads, 1930.

DEPARTMENT BULLETINS

No *136D. Highway Bonds. 20c.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
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*691D. Typical Specifications for Bituminous Road Materials. 10c.
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1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

No 331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No. 55T. Highway Bridge Surveys.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *914Y. Highways and Highway Transportation. 25c.
937Y. Miscellaneous Agricultural Statistics.
1036Y. Road Work on Farm Outlets Needs Skill and Right Equipment.

MISCELLANEOUS CIRCULARS

No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.
*93M. Direct Production Costs of Broken Stone. 25c.
109M. Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails, Flood Relief, and Miscellaneous Matters.

MISCELLANEOUS PUBLICATIONS

No. 76MP. The Results of Physical Tests of Road-Building Rock.

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)

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.

Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.

Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.

Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.

Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID AND EMERGENCY ROAD CONSTRUCTION

AS OF

MAY 31, 1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION						BALANCE OF FEDERAL AID FUNDS AVAILABLE FOR NEW PROJECTS	BALANCE OF EMERGENCY ADVANCE FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Federal Aid Allotted	Emergency Advance Fund	Mileage			Estimated Total Cost	Federal Aid Allotted	Emergency Advance Fund	Mileage				
					Initial	*Stage	Total				Initial	*Stage	Total		
Alabama	2,113.5	$ 7,060,353.62	$ 3,447,247.65	$ 1,370,863.51	208.1	117.0	325.1	$ 586,899.52	$ 293,449.73	$ 277,965.46	28.7	6.1	34.8	$ 3,355,041.20	$ 49,816.04
Arizona	863.3	5,151,196.04	3,422,908.64	999,074.98	200.9	182.9	383.8	1,484,583.29	1,107,244.85	155,686.98	62.7	30.3	93.0	159,568.69	1,199.89
Arkansas	1,746.4	8,450,676.29	4,030,656.68	1,280,157.00	220.8	92.1	312.9	1,492,194.39	716,448.26	128,000.00	36.7	16.9	53.6	448,154.04	
California	1,858.4	11,744,703.79	4,977,232.15	1,700,711.71	292.7	44.5	337.2	2,541,941.89	1,206,048.85	648,773.52	65.5	55.1	120.6	1,093,361.55	758,747.77
Colorado	1,326.4	5,216,229.84	2,764,233.03	987,050.51	160.2	89.0	249.2	1,088,611.86	607,750.27	306,109.40	54.7	21.3	76.0	2,183,653.50	214,672.09
Connecticut	257.3	4,643,863.46	1,756,735.33	520,491.00	43.9		43.9	775,572.90	244,161.06		10.8		10.8		
Delaware	306.0	1,041,247.28	519,869.13	400,000.00	52.6		52.6	149,652.50	74,826.25		5.9		5.9	88,702.46	
Florida	540.0	5,376,037.02	2,539,240.48	562,161.36	142.3		142.3	1,148,982.72	507,564.95	516,398.39	33.2		33.2	1,616,531.14	7,878.25
Georgia	2,756.2	8,846,394.45	4,296,367.89	1,880,127.33	316.8	101.8	418.6	2,846,258.37	1,307,709.43	197,196.62	106.6	34.1	140.7	1,060,913.36	673.06
Idaho	1,289.3	3,570,306.01	2,043,140.06	603,149.65	238.4	21.9	260.3	1,066,237.56	564,147.14	374,155.48	67.8	23.0	90.8	691,278.37	30,729.87
Illinois	2,218.7	24,224,950.88	11,102,567.56	1,751,000.00	711.2	43.4	754.6	8,587,568.66	3,858,112.12	1,644,000.00	281.1	4.7	285.8	1,499,724.64	5,116.00
Indiana	1,579.0	9,444.177.22	4,623,370.89	1,149,797.10	262.4		262.4	3,414,004.11	1,555,560.92	730,937.83	107.5		107.5	565,897.04	165,194.07
Iowa	3,153.0	7,405,514.83	3,136,700.61	1,963,369.00	167.2	73.3	240.5	635,822.75	218,775.38	153,000.00		20.2	20.2	4,059.10	
Kansas	3,120.4	7,561,353.77	3,619,066.59	952,453.36	446.9	38.6	485.5	2,898,251.90	1,391,113.94	415,507.95	148.6	40.1	188.7	665,061.21	786,918.35
Kentucky	1,549.3	6,724,260.50	2,950,303.66	841,965.71	263.2	90.8	354.0	3,167,235.48	856,551.39	615,990.08	64.9	74.2	139.1	207,441.93	46,759.21
Louisiana	1,422.9	8,935,142.75	4,115,439.52	1,132,343.87	269.6	17.3	286.9	122,208.27	58,515.39	10,000.00	5.1	.6	5.7	216,258.51	5,583.13
Maine	593.4	3,230,988.21	1,142,584.77	426,544.24	59.7		59.7	2,117,279.53	885,158.13	289,254.76	64.7		64.7	811,458.13	
Maryland	704.4	1,306,803.58	586,755.89	501,086.36	28.0	4.3	32.3	699,210.72	349,605.36	150,136.41	35.3		35.3	143,489.16	27,529.23
Massachusetts	724.3	8,270,987.53	1,979,587.66	1,141,460.00	68.8		68.8	1,282,505.26	420,709.94		14.8		14.8	2,019,371.02	
Michigan	1,804.8	9,674,682.80	4,029,607.21	846,000.00	265.9	22.4	288.3	1,936,851.96	796,288.00	386,000.00	54.3	15.7	70.0	2,804,888.98	1,289,382.00
Minnesota	4,107.4	3,925,900.10	1,614,177.75	845,000.00	44.6	164.4	209.0	5,034,424.68	2,175,498.19	1,284,993.00	35.3	167.9	203.2	21,519.47	120,000.00
Mississippi	1,772.7	3,806,686.27	1,872,537.77	1,351,844.54	158.5	80.5	239.0	502,078.19	251,039.08	42,343.16	17.2	16.4	33.6	3,851,023.96	40,548.30
Missouri	2,628.9	9,149,107.62	3,552,836.96	1,328,296.69	217.5	63.8	281.3	2,508,014.03	1,060,797.97	1,051,155.93	68.3	29.6	97.9	840,461.83	147,371.38
Montana	1,841.1	11,705,509.89	6,589,007.79	1,549,535.87	900.5	92.8	993.3	1,291,653.90	726,066.16	122,394.13	83.7	94.3	178.0	2,013,269.58	
Nebraska	3,815.4	8,667,914.12	4,059,639.34	1,432,333.74	232.4	204.8	437.2	2,222,492.68	1,045,506.62	275,697.26	75.7	84.4	160.1	1,077,770.23	
Nevada	1,184.1	2,315,183.89	1,593,829.83	549,296.82	74.2	156.5	230.7	1,200,014.54	726,439.96	429,490.07	33.8	121.3	155.1	350,347.84	70,851.11
New Hampshire	393.9	715,470.36	281,216.41	56,216.83	10.5		10.5	491,074.31	204,099.02	208,783.17	13.7	1.0	14.7	252,439.51	135,000.00
New Jersey	555.2	6,209,545.44	1,794,968.33	1,103,770.59	74.7		74.7	144,716.24	3,420.00		.2		.2	1,039,898.43	4,036.41
New Mexico	1,909.4	6,897,687.96	4,185,311.84	1,237,875.95	260.5	149.1	409.6	195,139.42	123,712.23	27,456.79	.2	10.6	10.8	151,097.44	37,955.26
New York	2,687.9	37,240,359.73	12,479,253.00	3,233,566.00	603.8	7.0	610.8	9,758,396.00	3,580,448.00	295,000.00	163.9		163.9	58,212.84	522,000.00
North Carolina	1,964.2	6,284,310.41	3,029,281.48	1,496,610.49	199.9	37.9	237.8	1,423,111.91	711,282.54	405,527.76	40.9	36.9	77.8	1,671,546.87	6,000.00
North Dakota	4,377.8	4,099,869.01	2,077,144.19	865,008.46	469.0	356.0	825.0	1,305,201.75	652,600.70	314,781.69	121.5	269.8	391.3	1,247,030.57	118,741.85
Ohio	2,548.1	14,342,823.96	4,534,966.20	1,279,205.55	213.3	17.7	231.0	5,702,551.70	1,705,713.81	1,713,332.45	82.6	19.3	101.9	1,918,069.66	6,000.00
Oklahoma	1,939.9	7,736,746.31	3,866,012.89	1,518,857.26	295.8	124.5	420.3	1,398,060.13	763,505.71	407,493.74	50.8	18.2	69.0	.02	
Oregon	1,259.5	7,381,069.21	4,116,307.31	795,397.31	224.2	93.4	317.6	1,369,755.96	678,312.31	520,008.59	77.0	1.8	78.8	396,256.10	
Pennsylvania	2,659.3	9,408,171.07	3,479,647.67	882,211.09	115.7		115.7	5,361,299.87	2,506,670.60	2,362,308.76	172.9		172.9	1,821,756.06	288,423.15
Rhode Island	209.8	2,939,304.60	1,118,710.80	400,000.00	47.8		47.8	91,465.74	45,732.86		.1		.1	72,870.99	
South Carolina	1,840.1	5,944,625.26	2,676,658.62	1,114,636.00	83.3	164.9	248.2							204,800.31	
South Dakota	3,718.2	5,532,393.30	3,069,749.40	1,153,688.39	435.4	161.2	596.6	466,287.54	231,404.61	127,675.51	25.1	35.2	60.3	857,861.61	52,609.10
Tennessee	1,458.2	4,527,947.60	2,236,446.06	1,656,863.16	191.9	13.4	205.3	478,499.69	239,249.83	48,929.00	34.9		34.9	1,859,028.31	36,089.84
Texas	6,984.5	18,649,260.75	6,478,317.80	4,445,451.02	800.2	291.2	1,091.4	3,347,992.78	1,593,384.36	642,628.98	130.7	47.6	178.3	3,985,839.70	
Utah	1,011.2	2,148,894.83	1,298,783.75	503,606.63	116.5	63.0	179.5	582,012.73	300,744.11	262,760.98	27.4	88.9	116.3	871,346.75	88,092.97
Vermont	301.6	1,230,278.38	504,939.26	311,064.10	23.5	4.8	28.3	194,608.31	91,496.71	60,275.93	7.1		7.1	22,073.06	28,659.97
Virginia	1,618.8	6,168,087.97	2,797,238.02	1,446,899.23	256.9	47.4	304.3	430,452.46	212,582.35	43,200.00	19.2	.6	19.8	729,721.02	15,402.77
Washington	1,001.0	5,120,210.06	2,290,609.05	976,854.93	170.7	19.8	190.5	859,509.86	353,885.90	256,842.79	16.8	12.5	29.3	1,197,006.02	37,235.28
West Virginia	768.2	4,636,470.21	1,789,818.30	409,585.46	109.5	12.5	122.0	1,575,547.62	618,272.31	429,399.49	34.5	12.9	47.4	319,716.56	36,399.06
Wisconsin	2,460.8	6,189,172.56	2,672,675.23	1,095,000.00	165.7	28.5	194.2	3,475,356.14	1,504,480.83	824,500.00	84.8	51.0	135.8	215,116.71	72,910.00
Wyoming	1,682.5	3,840,757.41	2,404,134.25	589,862.90	270.0	172.3	442.3	687,366.57	347,077.89	272,896.78	38.6	93.8	132.4	482,138.01	166,623.32
Hawaii	46.3	1,453,155.03	597,697.83	375,983.99	42.4		42.4	82,324.99	22,869.00		1.5		1.5	1,581,319.77	24,016.01
Totals	88,763.0	356,145,761.97	158,144,532.70	54,974,327.69	11,228.5	3,476.7	14,705.2	90,221,263.38	39,495,015.02	18,428,987.83	2,707.3	1,556.3	4,263.6	48,744,393.26	5,445,164.72

(*) The term stage construction refers to additional work done on projects previously improved with Federal Aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.