DISCUSSION ON SOIL WATER PHENOMENA

MR. BERT MYERS, *Iowa State Highway Commission.*

SUBGRADE MOISTURE CONTENT

For the past five years the Iowa State Highway Commission has been measuring the density and moisture content of subgrades supporting flexible pavements These pavements consist of a stabilized base course with a bituminous surface treatment.

These observations have been made at various seasons and at various elevations in an attempt to determine the maximum moisture content of these soils and the time of year at which this maximum occurs

The moisture content has been found to vary within wide limits depending on the type and density of the soil, on its position and on the season of year

The highest moisture content observed so far in the upper one foot of a subgrade was 33 7 per cent. This sample was taken at an elevation 5 to 6 in. below the upper surface of the subgrade on March 20, 1937. The soil was a silty clay loam containing 74 per cent silt and 25 per cent clay having a liquid limit of 41 and a plastic limit of 24 On December 8, 1938 this soil had a moisture content of 24 4 per cent or less than three-quarters of the maximum observed

In general it has been found that the highest moisture content may be expected in Iowa in the spring at about the time the soil has thawed.

It has been suggested that moisture may

TABLE 1

SOME OBSERVATIONS OF MOISTURE CONTENT OF SUBGRADE SOILS IN IOWA

GRUNDY COUNTY, ROAD NO 214, WELLSBURG SOUTH TO ROAD NO 14

Depth below bottom of base	Date sampled	Moisture content, per cent						
		No 274	No 282	No 290	No 298	No 306	No 310	Ave
in								
0–12	July '36	16 2	20 2	25 6	23 2	25 0	21 0	20 2
0–12	Apr '37	19 6	21 9	24 2	24 3	27 0	25 6	23 8
0– 6	Apr '37	11 6	16 3	12 6	18 9	25 9	23 7	18 2
0– 6	Nov '38	16 4	16 9	7 4	12 3	18 4	9 5	13 5

POTTAWATTAMIE, HARRISON, SHELBY COUNTIES ROAD NO 191, PORTSMOUTH TO NEOLA

Depth below bottom of base	Date sampled	Moisture content, per cent								
		No 492	No 493	No 496	No 509	No 508	No 506	No 505	No 504	Ave.
in										
0–6	Mar '37	25 3	16 8	19 5	20 4	15 9	20 5	20 2	15 7	20 3
0–3	Dec '38	20 6	15 3	16 9	24 4	21 4	19 6	20 2	16 6	19 4

CASS, POTTAWATTAMIE COUNTIES ROAD NO 83. ATLANTIC TO WALNUT UPPER 6 IN OF SUBGRADE

Date sampled	Moisture content — 19 locations		
	Min	Max	Ave
Dec '38	11 7	20 7	17 4
May '39	12 2	20 8	16 3
Sept '39	12 7	21 0	16 8

be concentrated in the soil at the upper surface of the subgrade or just below the lower surface of the base course.

The tests made do not indicate that this is true. In 285 cases observed the moisture content of the upper part of the subgrade was dryer than the next lower part in 211 cases or 74 per cent of the total. In the other 26 per cent of the cases the lower increment contained the same or less water than the soil on which the pavement rested.

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THE FLOW OF WATER IN SATURATED COHESIONLESS MATERIALS

Supplementary to the excellent contributions by M. B. Russell, M. G. Spangler and Bert Myers, the following material on the effect of soil water phenomena under specifically assumed conditions is given.

At preceding meetings of the Board, the upward flow of water in soil and quicksand was demonstrated by the writers. It is now proposed to extend the previous discussions; and to summarize the practical significance of the demonstrated phenomena with respect to the stability of highways. Specifically concerned are the possibilities of drainage with reference principally to cohesionless materials.

Suggested designs of adequate drainage systems are not included. Instead an attempt is made to identify the kinds of soil moisture which are amenable to drainage, and to present a clear picture of the stability of soil as affected by the moisture which cannot be removed by drainage.

The term cohesionless refers to granular materials and soil in which the ratio of film to free moisture in the mixture is too small to impart the properties of stickiness or cohesion. Cohesive, on the other hand, is used to designate those soils in which the ratio of film to free moisture is sufficiently high to impart cohesion to the soil-water mixtures. This is in accordance with material presented by the authors at a meeting of the Board three years ago (1)[1].

Materials which have affinity for water will be considered "hydrophilic; and for the purpose at hand, those which are water repellent will be termed "hydrophobic."

The effect of moisture, when conforming to the laws of water at rest is referred to as "hydrostatic," and when conforming to the laws of water in motion, as "hydrodynamic."

SUPPORTING VALUE OF SOIL AFFECTED BY ITS WEIGHT

An equation for determining the supporting value of soil beneath a strip load was published in the Proceedings of the Annual Meeting of the Board held two years ago (2).

The expression was derived on the basis of Coulomb's classical theory of the distribution of pressures in earth masses; and, for cohesionless soil is as follows:

$$q = WB\frac{\tan^4 a - 1}{\cot a}$$

where

- q = supporting value of the soil in pounds per square foot
- W = weight of soil per cubic foot
- B = ½ the width of the loaded strip
- a = 45° + ½ the angle of internal friction, ϕ, of the soil

The value, q, it is noted, depends upon functions of the angle, a, and is proportional to the values of W and B. It may be assumed that the value of a depends upon the porosity of the soil mass, and the size, shape and surface characteristics of the constituent particles.

The weight per cubic foot or density of the mass depends upon the porosity and the moisture content of the soil, and the specific gravity of the individual particles.

[1] Figures in parentheses refer to list of references at end

For soils with equal porosity and moisture content and particles of equal size, shape and surface texture, the supporting value of areas similar in size and shape, therefore, should vary directly with the specific gravities of the soil particles.

The specific gravity of sand grains, for illustration, is about 2.65; and of chemical lead, according to the Handbook of Chemistry and Physics published by the Chemical Rubber Company, about 11.0 or slightly more than 4 times that of the sand grains.

With the other variables kept constant then, lead shot should have at least 4 times the supporting value of sand

D. S. Berry found this to be true in laboratory tests (3). With a bearing area of 4 sq. in. and a settlement of 0 008 in., he found for smooth Ottawa sand, a supporting value of 864 lb. per sq ft. For No. 12 lead shot (1 27 mm.) with extra finish, it was 3,530 lb. per sq ft, or 4.08 times as much

EFFECTIVE WEIGHT OF THE SAME MATERIAL CAN BE VARIED WIDELY

Assume a cohesionless sand with constant porosity in different states or subjected to different hydrostatic or hydrodynamic effects. Consider five possible conditions as follows:

For condition I, take dry sand. Its density or weight per cubic foot depends entirely upon the porosity of the sand mass and the specific gravity of the sand particles.

For condition II, assume the material placed in the quicksand device (see Fig. 1-A); and water forced upwards through the pores of the sand. This dynamic action of the upward flow effectively reduces the weight of the particles; and, as a result, as has been shown at previous meetings of the Board, causes the sand to lose all semblance of stability. Consequently the small metal weight in the quicksand device (see Fig. 1-B) sinks abruptly to the bottom of the sand layer.

A small ball or sphere kept suspended in the air continuously by a water jet in exhibit windows illustrates the effect of the dynamic action of water in neutralizing the effect of gravity on the suspended particles

For condition III, let the water in the quicksand device be maintained at a constant elevation above the top of the sand The effect of the water in this case is hydrostatic; and it causes the submerged sand to have only a so-called "buoyed" weight; that is, the weight of a cubic foot of soil (solids + water) minus the weight of a cubic foot of water.

For condition IV, let the water be drawn downward through the sand in the quicksand device. The material is still completely immersed but, due to the dynamic effect of the downward flow of the water, has an effective weight greater than the buoyed weight of the sand.

For condition V, assume that the lowered water surface is maintained at the elevation of the bottom of the sand in the quicksand device. Columns of capillary moisture will then be sustained in the pores of the sand causing the particles still to be completely immersed Again, however, the effective weight of the sand exceeds its buoyed weight due to the hydrostatic condition termed capillary saturation.

P. C. Rutledge of Purdue University has suggested formulas for use in computing the effective weights of the sand for the five conditions just described (4).

The equations are as follows:

Condition I	Dry sand	$W = \frac{G}{1+e} 62\,4$
Condition III (hydrostatic)	Submerged sand	$W = \frac{G-1}{1+e} 62\,4$
Condition V (hydrostatic)	Saturated with capillary moisture	$W = \frac{G+e}{1+e} 62\,4$

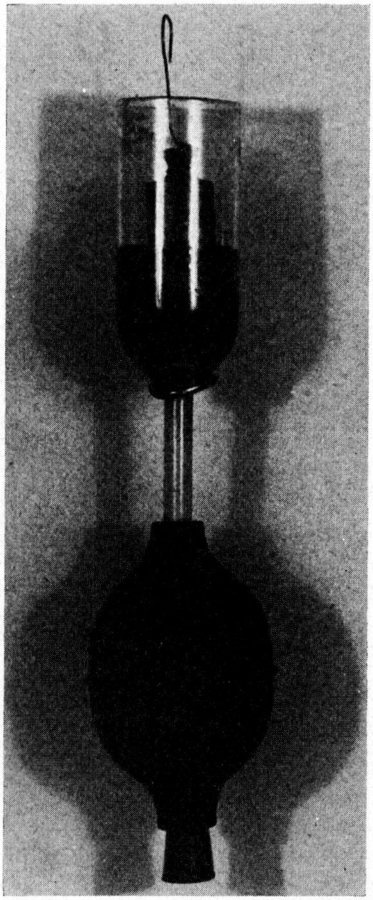

Figure 1A. Device for Demonstrating the Phenomenon of Quicksand. It Consists of a Small Glass Funnel (Carbon Filter Tube) Inserted in a Rubber Bulb. A Perforated Disk Is Placed in the Bottom of the Funnel, Water is Poured in Both Funnel and Bulb, Sand Is Placed in the Funnel on Top of the Perforated Disk and a Small Weight Is Set on the Sand. By Manipulating the Bulb, Water Can Be Made to Flow Either Upward or Downward through the Sand; or Maintained at Elevations Above, Below or Within the Sand Layer.

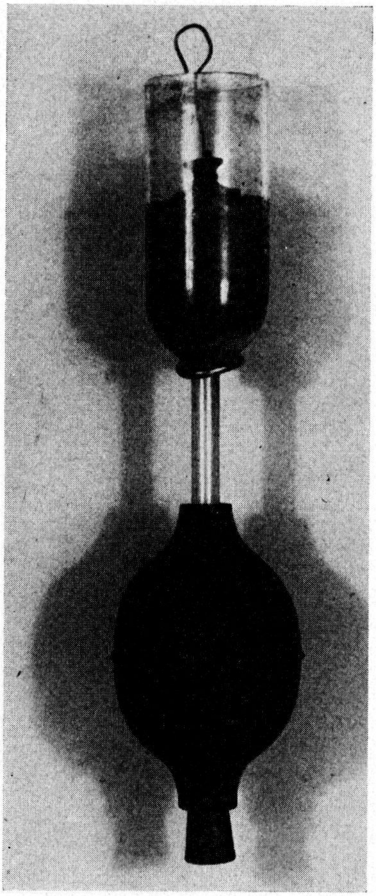

Figure 1B. Same as Figure 1A Except That the Small Weight Now Rests on the Porous Filter at the Bottom of the Sand Layer.

Condition II (hydro-dynamic)	Subjected to upward flow	$W=\left(\frac{G-1}{1+e}-H_g\right)62.4$
Condition IV	Subjected to downward flow	$W=\left(\frac{G-1}{1+e}+H_g\right)62.4$

In these equations,

W = weight of the sand per cubic foot
62.4 = weight of water per cubic foot
G = specific gravity
e = voids ratio
H_g = hydraulic gradient

$$=\frac{\text{change in head}}{\text{distance in soil}}$$

To obtain numerical values from these expressions, let $G=2.65$; $e=0.65$ and for the quicksand effect, condition II, $W=0$. Then

$$0=\frac{2.65-1}{1+0.65}-H_g$$

and $H_g=1$, which is an average value.

Making use of the numerical values of G, e and H_g as given, the computed effective weights of the sand become as follows:

Condition	Effective weight lb per cu ft	Water action
II—Upward flow	0	Hydrodynamic
III—Submerged	62.4	Hydrostatic
I—Dry	100.2	None
IV—Downward flow	124.8	Hydrodynamic
V—Capillary saturation	124.8	Hydrostatic

quicksand device, the capillary columns may be sustained when the free water surface is lowered through completely immersed soil. In either case, the length of the capillary columns sustained, or the height of capillary rise as commonly termed, is determined by a condition of equilibrium between the surface tension of the liquid and the weight of liquid in the capillary columns.

That capillary moisture is not likely to rise in hydrophobic or water-repellent materials can be demonstrated by means of two glass tubes with small bore. Let the bore of the first tube be uncoated and the bore of the second tube be coated with paraffin. There will be a rise of the capillary moisture column if one end of the first tube is immersed in water. Under similar conditions of immersion, however, water in the second tube will be depressed below the elevation of the water in which the end of the tube is immersed.

The apparatus used by the Public Roads Administration for determining the capillary rise of relatively cohesionless materials has been described previously in the Proceedings of the Board (5). It is a modification of the Bartell cell, originally proposed in 1896 for use in determining the capillary characteristics of lime and cement mortars.

For demonstration purposes, use of the simplified device, shown in Figure 2 is convenient. Assume that it contains the same granular material as was used in the quicksand device. First, the sand is inundated by lowering the funnel carefully into the water of the beaker. Then the funnel is slowly raised. So long as the

CAPILLARY PRESSURE INCREASES WITH THE LENGTH OF THE SUSTAINED COLUMNS OF CAPILLARY MOISTURE

Capillary rise in soil is explained as follows. To begin with the particles must be hydrophilic. Attraction of the sand or other material for water then causes minute moisture films to coat the contained pores far above the elevation of the water supply. Due to the phenomenon of surface tension of the liquid, the capillary columns are drawn upward through the pores by the rising moisture films. On the other hand, as was the case in the column of water in the glass tube forming the lower part of the funnel beneath the sand remains unbroken, it is considered that the capillary rise of the material has not been reached.

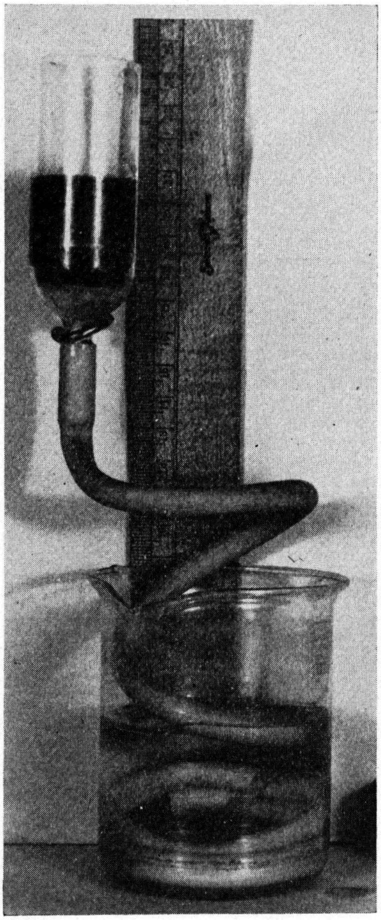

Figure 2. Capillometer Suitable for Purposes of Demonstration. It Consists of a Carbon Filter Funnel with Perforated Disk Like That Used in the Quicksand Device and Attached to a Rubber Tube as Shown. Use of the Rubber Tubing Permits the Elevation of the Funnel with Respect to the Water Supply in the Beaker to Be Changed Rapidly and Conveniently.

Until the capillary rise has been exceeded, the sand not only has weight, but also on its surface, contributing further to the stability of the material, a capillary pressure is acting, which is equal to the product of the distance between sand surface and the surface of the water supply in the beaker, multiplied by 62.4 lb., which has been given as the weight of water per cubic foot. For columns 1 ft. long, the pressure exerted on the surface of the soil is 62.4 lb. per sq. ft.; for columns 2 ft. long, it is 124.8 lb. per sq. ft.; and for columns 3 ft. long, it is 187.2 lb. per sq. ft.

Now refer to the computed densities of the sand given above. The values of W, it will be noted, are equal for both the conditions IV of capillary saturation and V of downward flow of the water. Therefore, at any depth, d, below the surface of the sand, the pressures, dW, produced by the effective weights of the sand should be equal. This, however, is true only in one case; namely, when the location of the computed pressures is taken at the elevation of the ground water.

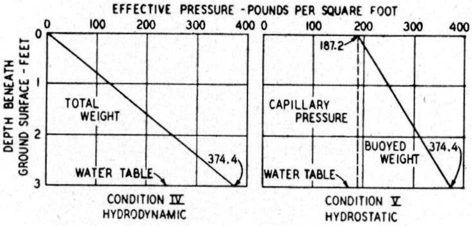

Figure 3. Diagram of Effective Pressure in Soil

To illustrate this pertinent fact, let the ground water elevation be 3 ft. beneath the surface of a level-deposit of cohesionless material, which has the porosity, etc., of the sand being discussed; and also a capillary rise of at least 3 ft. For both conditions IV and V, the vertical earth pressure at the depth, d = 3 ft., is the same and equals $3 \times 124.8 = 374.4$ lb. per sq. ft. However, for all other locations within the zone of capillary saturation, as illustrated in Figure 3, the pressure for condition V exceeds that given for condition IV.

The reason for this is apparent. The pressure for condition IV begins with a value of 0 at the surface and increases proportionately with depth at the rate of 124.4 lb. per sq. ft. until the value of 374.4 lb. is reached at the depth of 3 ft. For condition V, in contrast, the pressure at the surface begins with a value of 187.2 lb. per sq. ft. produced by the capillary columns 3 ft. long, and then increases proportionately with depth at the rate of 62.4 lb. (buoyed weight) per sq. ft. per ft. until at the depth of 3 ft. the value of 374.4 lb. per sq. ft. is also attained.

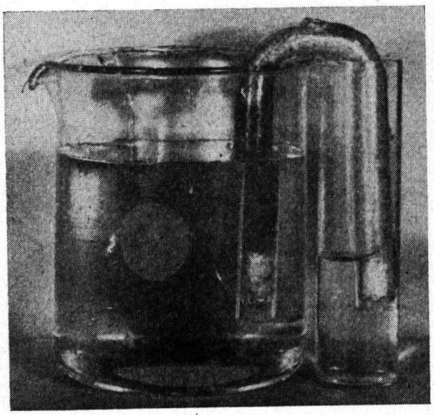

Figure 4. Beakers and Sand-Filled Glass Tube Suitable for Demonstrating the Phenomena of Flow in Cohesionless Soil.

For the five conditions considered, therefore, within this depth, the effective weights of the sand may be varied from 0 to 124.8 lb. per cu. ft., with a possible pressure addition on the upper surface for condition V. With the highest weights the sand is most stable. Therefore, it has the highest supporting value when the flow of water is downward, and when the sand pores contain only capillary moisture.

CAPILLARY RISE AND FLOW OF GRAVITATIONAL MOISTURE RELATED

The flow of water through cohesionless soil can be demonstrated by the apparatus, Figure 4. It consists of two beakers and a bent glass tube. Let the tube be filled with dry Ottawa sand and have a cotton wad inserted at each end to prevent the sand from escaping. Place water in only the larger beaker.

If one end of the tube is immersed in the water of the larger beaker, and the other end is placed in the smaller empty beaker, the water will rise in the sand grains, pass through the curved part of the tube over the edge of the beaker, and flow downward into the smaller beaker. The flow will continue until the water in

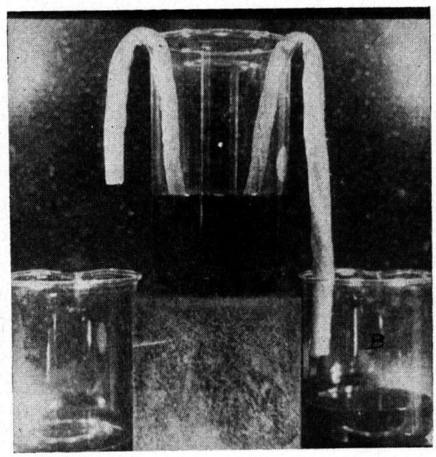

Figure 5. Beaker and Wick Arrangement for Demonstrating the Phenomena of Flow in Cohesionless Materials—from *Public Roads,* June 1931.

both beakers attains the same elevation. A bent glass tube of small capillary bore or a lamp wick can be used to illustrate the same phenomena.

In like manner, one end of a napkin immersed in thin soup can, if kept there long enough, cause an embarrassing transfer of the liquid from the soup bowl to the table cloth; or one end of a towel left carelessly immersed in a bath tub can cause a damaging transfer of the bath water to the bath room floor.

Similar phenomena were described in *Public Roads,* June 1931. The apparatus consisted of three glass beakers and two cheese cloth wicks arranged as shown in Figure 5. The liquid from beaker A rose in both wicks, passed through them over the edge of container A, and then proceeded down to the ends of the wick outside of the container.

However, the amount of moisture observed in the two wicks differed. The wick on the left whose outside end was slightly above the elevation of the surface of the liquid in container A became only moist. On the other hand, the wick on the right whose outside end extended below the elevation of the liquid in container A, became dripping wet, and drop by drop transferred the liquid from beaker A to beaker B.

Obviously the rise of the water as described is due to capillarity. Therefore, such water as rises in the sand-filled tube and wicks is truly capillary moisture. But it has two characteristics which are not generally attributed to capillary moisture.

In the first place, this type of capillary moisture freezes at the normal freezing temperature of water in bulk If placed in a mechanical refrigerator at ordinary freezing temperatures, both the saturated sand and the wicks can be frozen solid.

Secondly, this moisture which was of the capillary type while rising inside of the larger beakers, became free or gravitational water on the outside of the beakers when the ends of the tube and wick (on the right) were placed at an elevation below the surface of the water in the beakers.

Therefore, it is considered that capillary moisture of this type conforms strictly to recognized concepts of surface tension, the force of gravity and the principles of hydraulics as applied to free water.

On this basis the formula commonly used for computing the height of rise in capillary tubes was derived.

The expression is as follows:

$$h = \frac{4 \cos a}{980\, p\, G_1} S_t$$

where

h = height of capillary rise in cm
p = diameter of the capillary tube in cm
S_t = surface tension of the liquid in dynes per cm
a = angle of contact between liquid and tube
G_1 = specific gravity of the liquid
980 = acceleration due to gravity

For water, G may be taken as 1, and S_t as 72 8 dynes per cm.,[2] and for h_{max} the value of a, the angle, becomes 0. By substituting these numerical values and multiplying by 10/2.54 to convert p to millimeters and h to inches, the equation becomes

$$h = \frac{1.17}{p} \qquad (1)$$

This relation, it will be noted, is shown by a straight line when both h and p are plotted to a logarithmic scale

Tests for determining the flow of gravitational water through soil are usually made in a variable head permeameter. Its essential features are a standpipe and a soil container arranged as shown in Figure 6.

The level of the tail water is kept constant and the resistance to flow in all parts of the apparatus is negligible in comparison with that of the soil sample. Therefore, the height of the water column at any instant, measured from the tail-water surface to the water level in the standpipe is the driving force causing permeation.

The test furnishes values of the coefficient of permeability of the soil, k. This coefficient indicates the quantity of percolation at an hydraulic gradient of 1. The expression used for determining, k, is as follows:

$$k = \frac{2\,3\, al}{A\,(t_2 - t_1)} \log \frac{h_1}{h_2} \qquad (2)$$

[2] For temperature of 20°C or 68°F, the surface tension of water against air is 72 75 dynes per cm.

where

- a = horizontal cross-sectional area of the standpipe
- A = horizontal cross-sectional area of the soil sample
- l = the height of the soil sample in feet
- h_1 = the effective head in the standpipe at a time, t_1
- h_2 = the effective head in the standpipe at a time, t_2

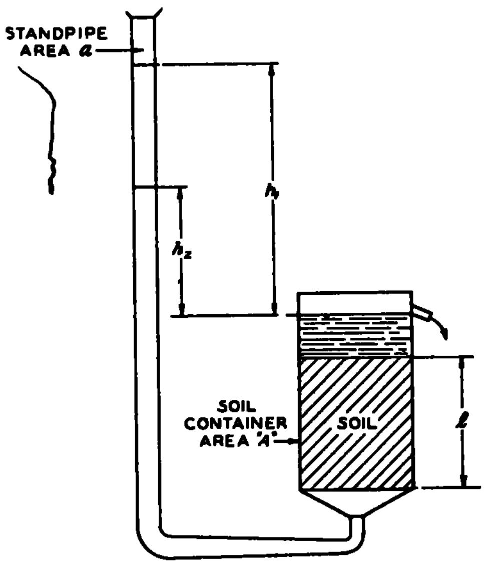

Figure 6. Schematic Diagram of Variable Head Permeameter

Now consider the data, Table 1, which were obtained from experiments made by the Public Roads Administration some years ago. The observed values of the capillary rise, h, were obtained by means of tests made in the capillometer referred to above The computed values of the effective pore diameter were obtained by substituting the observed values of, h, in equation 1 and solving for, p. The values of the coefficient, k, were obtained from tests made in a variable head permeameter which had the essential features shown in Figure 6 and by the use of Formula 2.

The relation of p to d is shown in Figure 7; of d to h, in Figure 8, and of h to k in Figure 9.

By combining Formula 1 with the formulas shown in Figures 7, 8 and 9, it is of interest to note that for the type of sand tested, the values of p, h and k can

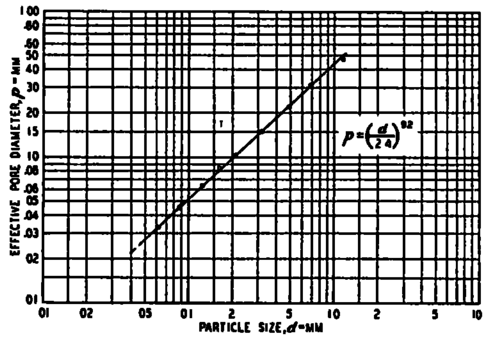

Figure 7. Relation Between Particle Size and Pore Diameter

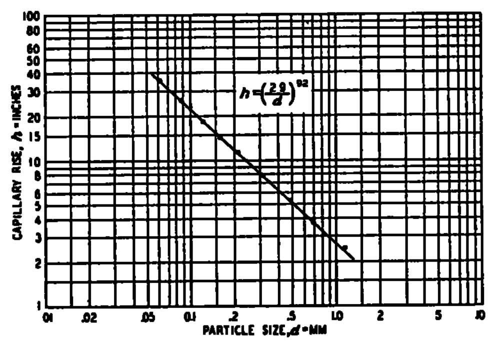

Figure 8. Relation Between Particle Size and Capillary Rise

be shown with respect to the average grain size, d. The expressions are

$$d = 2.4\, p^{1.087}$$

$$d = \frac{2.9}{h^{1.087}}$$

$$d = \frac{k^{.625}}{87.1}$$

The existence of such relations affords a means of very materially reducing the testing efforts required for particular purposes. To illustrate, let it be assumed that the relation of k to h has been determined

TABLE 1
FLOW CHARACTERISTICS OF DIFFERENT FRACTIONS OF RIVER SAND

Fraction: Passing	Fraction: Retained on	Mean grain diameter d[a]	Capillary Rise, h	Computed pore diameter p[b]	Coefficient of permeability k
Sieve No	Sieve No	Mm	Inches	Mm	Feet per day
10	20	1 183	2 48	0 472	1430
20	30	0 693	3 74	0 313	665
30	40	0 491	5 24	0 223	380
40	60	0 313	7 88	0 149	190
60	80	0 207	11 7	0 100	160
80	100	0 162	14 0	0 084	75
100	140	0 123	18 5	0 063	45
140	200	0 087	26 4	0 044	20
200	270	0.062	35 6	0 033	9

[a]The expression used in making computations of the mean grain diameters is

$$d = \frac{2\, d_1 d_2}{d_1 + d_2}$$

where

d = mean grain diameter.
d_1 = sieve opening of the sieve which the fraction passes.
d_2 = sieve opening of the sieve on which the material is retained.

Thus, for the fraction passing the No 10 sieve and retained on the No. 20 sieve

$d_1 = 2$ mm.
$d_2 = 0\ 84$ mm.

and

$d = 1\ 183$ mm

[b] Computed by means of formula 1.

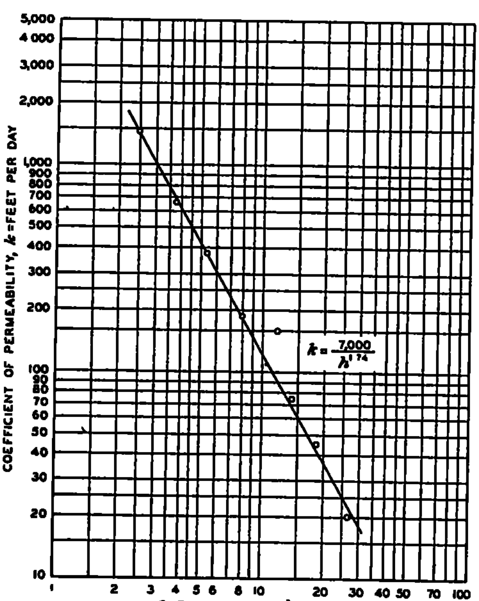

Figure 9. Relation Between Capillarity and Permeability

for particular soil types of which an earth dam is to be constructed; and furthermore, that material for different parts of the dam are to be selected on the basis of their permeability. Under these conditions, a relatively simple capillary test suitable for use under field conditions, may then be employed on the job to determine the coefficients of permeability of the borrow pit materials

A simple test of this type for cohesionless soils (principally silts) was developed by Arthur Casagrande for control purposes in the construction of the Granville dam, New Hampshire. The test has been described elsewhere under the caption, horizontal capillary test (6) (7).

Use of the straight line plats, permits ready extension of the relations to particle sizes not included in the tests. For illustration, consider the silt fraction of soil, which consists of particles ranging in size from 0 05 mm. to 0.005 mm.

Elsewhere (8) it has been stated that "Silt usually consists of bulky grains, similar except for size to fine sand and having the same mineral composition." Also, like sand, silt has little or no cohesion.

Therefore, it seems reasonable to assume that the relations established for the sand fraction of the soil tested are valid also for the silt fraction. If this is true, the relations shown in Figures 7, 8, and 9 can be extended at least to the value of d=.005 mm., the minimum particle size

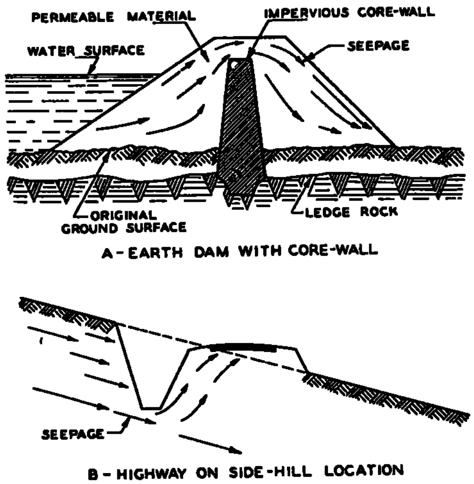

Figure 10. Illustrations of Flow Phenomena in the Field. From *Public Roads,* June 1931.

of the silt fraction Such extensions on charts with greater scale range than the ones shown give values for d=.005 mm. as follows:

p=0 0033 mm , h=355 inches, and k=0 2 foot per day.

How far, if at all, the relations can be extended into the realm of cohesive clay fractions is not known at the present time.

EFFECT OF FIELD CONDITIONS CONSIDERED

Figure 10 reproduced from *Public Roads,* June 1931, illustrates the occurrence of capillary moisture and seepage under field conditions. The flow indicated in Figure 10-A conforms to that demonstrated by the wick and sand-filled tube experiments previously described.

Despite the extension of the impervious core wall above the elevation of the adjacent water surface, capillary moisture may penetrate the pervious material above the wall and, dropping down on the other side, produce the syphon effect which results in seepage at the lower face of the dam. In a similar manner capillary moisture can rise in subgrades despite the construction of cutoff ditches as shown in Figure 10-B.

The capillary rise and the permeability of soils as determined for the idealized conditions represented in tests performed under rigid laboratory control indicate the extent to which the phenomena illustrated in Figure 10 are likely to occur under field conditions. However, the amount of damage likely to be caused by capillary moisture and seepage is influenced also by numerous other variables such as topography, soil profile, climate, type of pavement or road surface and traffic. Limitations of this discussion permit consideration of only the evaporation of moisture from the ground, and the atmospheric variables of humidity and temperature at this time.

EFFECT OF HUMIDITY AND EVAPORATION SUMMARIZED

The rate of evaporation greatly affects the capillary moisture of soil at or near the earth's surface. This in turn is appreciably influenced by the humidity of the atmosphere, being greater when the humidity is relatively low than when it is high. Therefore, humidity and evaporation are considered jointly.

Evaporation from soil covered with porous material is more rapid than from similar soil covered with impermeable material Therefore, the use of porous material in shoulders affords a means of expediting the escape of moisture which reaches the subgrade due to capillarity. Making the porous shoulders continuous with subbases which provide for aeration beneath pavements adds additional benefit However, such porous layers, must be continued to side drains or ditches to prevent their serving as entrances for water from rains and thaws rather than exits for offending moisture

Vegetation is also an influencing factor. The evaporation which takes place from the surface of every blade of grass and the leaves of all trees and bushes exerts a two-fold effect. In the first place, it expedites the escape of moisture from the ground. Secondly, it causes the humidity in the vicinity of the vegetation to remain more or less constant The combined result of these effects plus that offered by possible shade, is to stabilize the moisture content of road soils

With the same conditions of capillary rise and evaporation from road surface and shoulders, subgrades located in areas of high humidity should have higher capillary moisture contents because of slower evaporation. Also, seepage resulting from capillary phenomena should be greater in areas of high humidity.

This was indicated by the wick experiment previously described. At the time the photograph (Fig. 5) was taken the humidity was relatively low and the measured discharge from the wick on the right was at the rate of 1/26 gal. per day. At other times during the experiment when the humidity was relatively high, the wicks appeared wetter and the wick on the right attained a discharge of about twice as much or 1/12 gal. per day.

FROST PHENOMENA SUMMARIZED

Damage due to frost which is the most spectacular effect of temperature variation, has been described repeatedly. For the present purpose, it suffices to say that ice lenses are involved and that two theories have been advanced to explain their formation

One theory was advanced by Stephen Taber (9) and the other by Benkelman and Olmstead (10)

The requirements for the growth of ice lenses according to Taber's theory are·

1. Soil with detrimental capillary properties

2. The presence of pore water (probably in film phase) in the soil, which has the property of remaining unfrozen at temperatures far below the freezing temperatures of water in bulk. Such moisture, it is indicated may remain unfrozen at temperatures as low as 70° below zero, C.

3. The occurrence of more or less continuous periods of normal freezing air temperatures; that is, below 32° F. or 0° C.

Acording to Benkelman and Olmstead, ice lenses may be formed under conditions as follows:

1. In relatively permeable soil without respect to its capillarity.

2. At complete saturation of the soil with pore water which can be frozen at the freezing temperatures of free water in bulk.

3. With alternate fluctuations of normal freezing and thawing temperatures.

Also, the resulting damage due to frost is of two kinds.

In one kind, moisture drawn from the water table below or other unfrozen source migrates towards the surface of the soil during freezing weather and forms the ice layers which produce considerable heave of road surfaces.

The second type of damage consists principally of an abrupt drop in the stability of soils during thaws. The road surfaces and pavements may not heave appreciably and are likely to remain undamaged as long as the undersoil remains frozen.

Under conditions of accelerated freezing in the laboratory, it has been found possible to produce ice lenses in every type of soil from the permeable sands to the colloidal clays.

However, this has not been found to be the case in the field. Under the normal climatic conditions of continental United States, it has not been found that plastic clay soils have been seriously affected by frost; and from investigations in Denmark and Sweden, it can be concluded that cohesionless soils, with a capillary rise not exceeding about three feet (11)[8] are not likely to prove troublesome due to frost.

It is the relatively cohesionless silt soils which have proven most troublesome; that is, the soils which, according to previous discussion have values of capillary rise ranging from about h=3 to at least 30 feet.

Observations of frost in such soils indicate that both of the theories mentioned above are valid. In some cases the water productive of damage undoubtedly rises from the water table due entirely to the capillarity of the soil. For damage to occur in other cases, it seems that the soil must first be completely saturated in much the same manner that a pump is primed before water can be raised from the well beneath.

EFFECT OF TEMPERATURE EXCLUSIVE OF FROST SUMMARIZED

On drop of temperature, the moisture films on soil particles increase in thickness, and the free pore water becomes more viscous. As a combined result of these influences, water percolates through soil in cold weather at a much slower rate than in warmer weather.

Conversely, the flow of soil moisture is abruptly accelerated by a rapid rise of temperature. Experiments performed by the Public Roads Administration indicated an increase of more than 100 per cent in the value of the coefficient of permeability, k, with rise of temperature of the soil from 35° F. to 85° F. (12) Water furnished by thaws is additive to the amount of water made available for flow by rise of temperature alone.

In relatively impermeable materials where rapid seepage cannot occur, the additional water thus released for flow by sudden rise of temperature causes an abrupt drop in the stability of the soil. This was indicated by other experiments of the Public Roads Administration (see reference 13). The stability of compacted clay samples, as indicated by the Proctor plasticity needle dropped from 1,400 to 960 lb. per sq. in when the temperature was raised from 42° F. to 130° F.

This drop of stability occurred without change in the moisture content of the soil and, therefore, was due to an hydrostatic effect of water or the effect of water at rest. On the other hand, water released by sudden rise of temperature in relatively permeable silts and sands produces an overabundance of movable water. The resulting loss of stability in these materials, therefore, can be attributed, at least in part, to the effects of water in motion or hydrodynamic effects.

HYDRODYNAMIC EFFECTS OF SOIL MOISTURE SUMMARIZED

It is the dynamic action of water which contributes to loss of stability due to the condition termed "sudden draw down" (13). To illustrate, let fine cohesionless sand be gently poured into a large container of water. The sand will form a mound with side slopes of about 1½ horizontal to 1 vertical. The immersed sand will maintain these slopes so long as no movement of the water occurs. If, however, the elevation of the water be quickly dropped, the rush of water from the interior sand pores

[8] Actually Gunnar Beskow gives a value of h less than 40 inches for soils which are not subject to frost heaving. He states further, however, "when the figures exceed these limits, the soil is usually classed as frost heaving, although those near the limits in reality belong to a rather complicated border zone."

will cause a floating of the particles on the surface of the mound with the result that the slope of the mound becomes flatter.

If the elevation of the water is raised and lowered enough times, the slope of the sand mound will become very flat. Relatively flat sand beaches illustrate the effect of long repeated draw down produced by wave action.

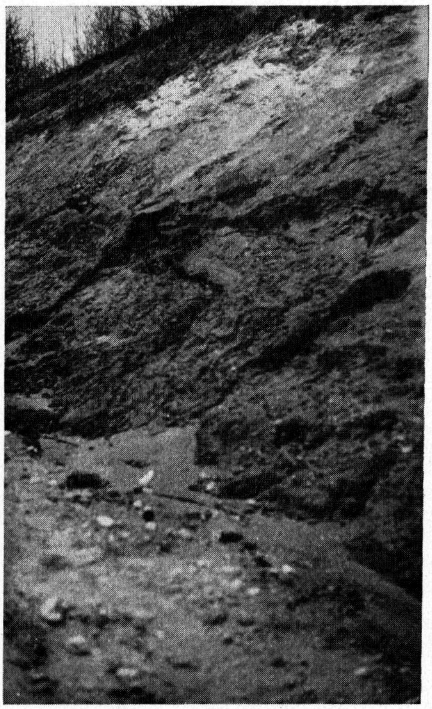

Figure 11. Eroded Face of Cut—from *Public Roads,* June 1931

In like manner, all attempts to dig holes in beaches are futile because adjacent sand fills the excavation area as quickly as sand is removed from it. Again, this occurs not because the sand is merely wet or saturated but because the sand grains are floated into the hole by water flowing from the pores of the sand.

A common procedure under these conditions is to surround the area to be excavated with well points connected with pumps to remove the damaging ground water. The distance of the well points from the excavation location may be selected so that the face of the excavation will receive the full stabilizing effect of capillary saturation. In this manner, excavations with almost vertical faces can be made close to the ocean without the use of shoring. However, this method has one disadvantage. Should unforeseen circumstances cause the pumps to stop, the excavated area will be quickly filled up by the migrating sand.

To a somewhat less extent, draw down is produced in earth dams by the sudden drop of water in reservoirs; and in dikes and levees by the rapid recession of flood waters.

The effect of seepage in relatively permeable materials is similar to sudden draw down. Accelerated seepage due to rise of temperature in the spring may cause the slopes of cuts to slough off as shown in Figure 11. Instances have been noted where seepage has so nearly floated materials at the lower face of a dam that one would sink into them up to the hips. One method suggested by a noted authority before the Board several years ago, for alleviating the unstable condition was to plant willows or similar vegetation which affords a rapid transfer of moisture from the ground to the atmosphere.

At times, moving water affects also the driving of piles in cohesionless granular materials. If the submerged sand exists in a highly porous state, the pressure delivered by blows of the pile driver on top of the pile, causes quick surges of water through the sand beneath and adjacent to the pile. The resulting quicksand effect causes an excessive penetration of the pile. Gradually, however, high resistance to further penetration of the piles is likely to develop, thus indicating that the sand foundation has been consolidated by the vibratory effects of the pile-driving action. Some engineers hold that attempts to obtain additional penetration in sand thus compacted may result in damage to the pile.

In addition to the localized slips and slides which damage the slopes of cuts and embankments, there is evidence that the uplift or quicksand effect may have much to do with landslides of very large proportions. As an illustration of this kind, Charles Terzaghi, at the Fourth Texas Conference on Soil Mechanics and Foundation Engineering, referred to a slide of very large dimension, which occurred periodically along the Straits of Dover, England. The conclusion reached from very extensive investigations was that the sliding of the otherwise stable area was caused by a ripid rise of the ground water at peak conditions of precipitation.

CLOSING COMMENTS

It is quite evident that soil moisture by both hydrostatic and hydrodynamic action can influence very radically the effective weight of soil and, therefore, the resultant stability of the soil mass.

Relatively simple tests, based upon sound principles of hydraulics, have been developed for determining the flow characteristics of moisture which affect the stability of soil.

In the application of test data, however, the effect of a number of variables present under field conditions but excluded in rigid laboratory control, must also be considered.

Apparently all moisture which produces frost damage in accordance with the theory advanced by Benkelman and Olmstead is drainable. The same is true of all moisture productive of hydrodynamic action and seepage.

It is obvious that capillary moisture cannot be removed by drainage. However, this type of moisture is not likely to be damaging either with or without frost to cohesionless soils, which have a maximum capillary rise not exceeding about three feet.

In cohesionless soils with a capillary rise of moderate heights exceeding three feet, it may be possible to lower the water table enough by drainage to prevent damage by the kind of frost considered in Taber's theory. Such procedure as a preventive of frost damage in cohesionless soils of relatively high capillary rise seems impractical.

If all the water that is amenable to drainage is removed from cohesionless soils, the remaining capillary moisture may, in the absence of frost, exert a stabilizing instead of a detrimental effect.

Therefore, it can be concluded that much of the water productive of damage in cohesionless soils is amenable to drainage. It now remains to determine the designs of drainage systems required to adequately accomplish this purpose. The final problem is economic and involves the cost of means used to remove moisture from soil adversely affected as compared with the cost of substituting other materials which will not be adversely affected.

The solution of this problem provides a wide field for future research.

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SOIL MOISTURE CONTENT AND DENSITY DATA

When a relatively large quantity of free water, a rainstorm for example, suddenly comes in contact with a dry, highly expansive clay soil structure, it causes the soil structure where wetted to disintegrate. The phenomenon might be compared to that of pouring boiling water on a cold egg. On the other hand recent laboratory experiments show that slow moisture content fluctuations did not break up any part of the soil structure.

EXPERIMENTS

In order to study the effects of slow moisture content changes, specimens of clay soils of various densities were subjected to three cycles of alternate wettings and dryings

A well mixed clay soil (Sample No SII) having an L.S. of 18 and a P.I. of 27 was used. Three groups of specimens ("A," "C" and "E") of four specimens each were molded in 16 cc monel metal pat cups so that the groups were produced with densities ranging from 89 to 129 lb. per cu. ft. when oven-dried at 100° F, as shown in Table 1, second column [1]

The specimens were first wetted slowly until each had absorbed all the water it would, at which time the moisture contents, densities, and volumes of the saturated specimens were determined. Then, the specimens were dried slowly to constant weight at 100° F. when the volumes and densities of the specimens again were taken. The specimens were carried through this cycle two times. The data are shown in Table 1. It is expected that these specimens will be wetted and dried through a sufficient number of additional cycles to determine whether the results obtained to date are constant

DISCUSSION

Perhaps the first mental reaction to the data is that clay soil under pavements seldom becomes as dry as these specimens were made The first reply is that the main objective in these experiments is to determine the trend of the differences in reactions in a soil compacted to different densities. Second, if it is found that the differences in the volumetric changes in soil compacted to different densities are discernible in the behavior of the overlying pavement or riding surface, then it is practicable to determine the densities, volumetric changes, etc, when the soil specimens are at intermediate moisture contents This was done with another clay soil, Sample No. SI, with an L.S. of 16 and P.I. of 22, which is now being tested in the same manner as the SII soil, as illustrated in Figures 1 and 2.

A summary of the SII soil data are

[1] Some of the specimens have been lost on account of breakage, etc.

TABLE 1

Group and Specimen No	Treatment of specimens after being molded wet								
	First drying	First wetting			Second drying	Second wetting			Third drying
	Density or weight lb per cu ft 0—moisture	100° F dry density or weight lb per cu ft	Moisture %	Vol change % of last 100° F oven-dry density	Density or weight lb per cu ft o—moisture	100° F dry density or weight lb per cu ft	Moisture %	Vol change % of last 100° F oven-dry density	Density or weight lb per cu ft o—moisture
1-SII-A-2	89 33	68 89	46 58	29 7	89 91	70 31	43 47	27 9	89 49
1-SII-C-1	106 80				108 08	80 68	32 28	34 0	106 52
2	110 15	82 32	33 06	33 8	110 62	82 72	31 30	33 8	109 67
3	112 77	81 84	31 88	37 8	113 43	84 21	30 41	34 7	112 14
4	112 25	82 00	32 13	36 9	112 65	83 37	31 02	35 1	111 15
Average	110 49	82 05	32 36	36 2	111 19	82 74	31 25	34 4	109 87
1-SII-E-1	129 20	95 12	23 93	35 8	130 77				
2	128 61	93 92	24 62	36 9	130 63	94 49	23 47	38 2	129 68
3	129 54	96 12	23 29	34 8	130 88	95 50	22 86	37 1	129 78
4	129 04	95 96	23 20	34 5	131 46	94 48	22 84	39 2	129 82
Average	129 10	95 28	23 76	35 5	130 93	94 82	23 06	38 2	129 76

shown in Table 2 upon which the following observations are made:

(1) As shown in Columns 3, 6 and 7, the first or original dry densities of the differently compacted groups of soil specimens were:

Group A— 89 lb per cu. ft.
Group C—110 lb per cu ft.
Group E—129 lb per cu ft.

Each time the specimens were wetted to saturation and oven dried to constant

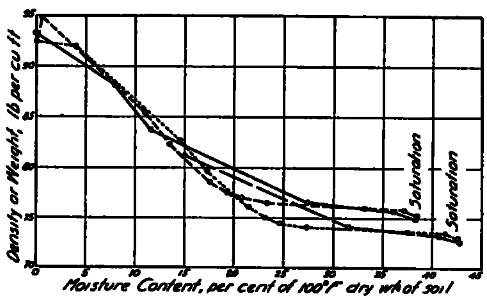

Figure 1

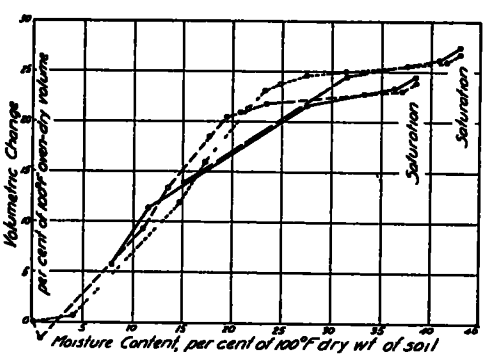

Figure 2

weight, the average densities were practically unchanged. These data also are shown graphically in Figure 3.

(2) As shown in Figure 3, there was no appreciable difference in the group average densities each time the specimens were wetted to saturation.

(3) As shown in Table 2, column 8, the lightest compacted group "A" specimen had absorbed the greatest amount of

TABLE 2

SUMMARY OF COMPILED DATA

1	2	3	4	5	6	7	8	9	10
Group No.	No of soil specimens in group	First time 100° F oven-dry in lb per cu ft.	No of times specimens were oven dried after being molded wet	No of times specimens were water saturated after being molded wet	Maximum variation in density or weight in lb per cu ft		Moisture content when saturated in per cent of dry wt of soil	Between oven-dry and saturated specimens	
					When 100° F. oven dry	When water saturated		Density change in lb per cu. ft	Volumetric change in per cent of oven dry volume
1-SII-A	1	89	3	2	1	1	45	20	29
1-SII-C	4	110	3	2	1	1	32	28	35
1-SII-E	3	129	3	2	2	0	23	35	37

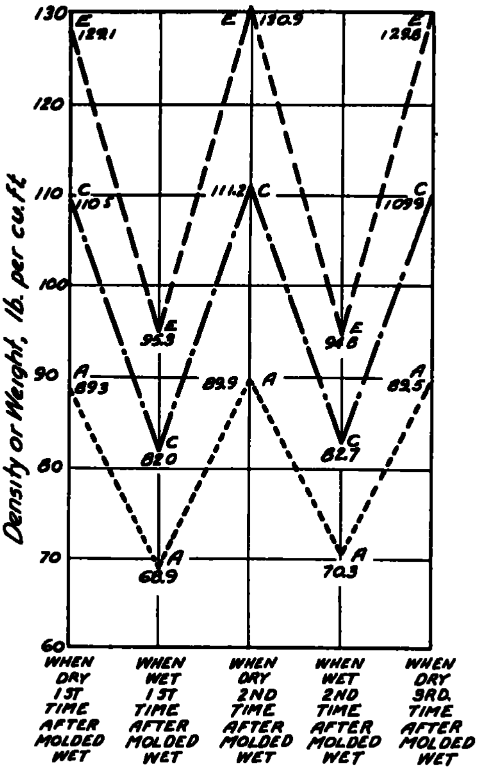

Figure 3

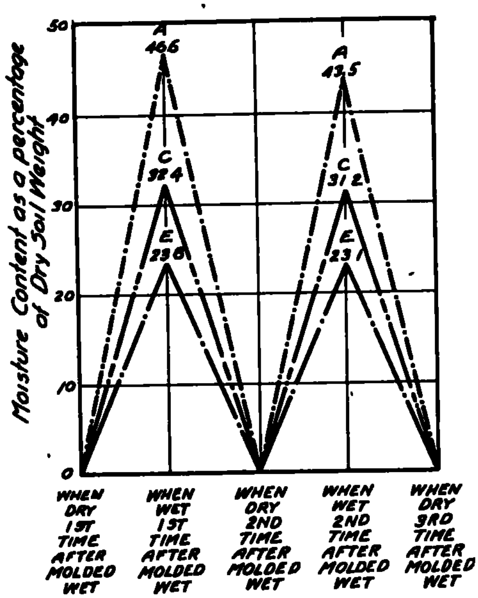

Figure 4

water when it reached saturation, 45 per cent. The intermediately compacted group "C" specimen had absorbed the next largest amount of water, 32 per cent; and the densest compacted group "E" specimens had absorbed the least amount of water when they reached saturation, 23 per cent. These data are shown graphically in Figure 4.

(4) On the other hand, as shown in Table 2, column 9, between the oven-dry and water saturated conditions of the specimens, the changes in density of the soil were·

Group A—20 lb. per cu. ft.
Group C—28 lb. per cu. ft.
Group E—35 lb. per cu ft.

These data are shown graphically in Figure 3.

(5) And as shown in Table 2, column 10, between the oven-dry and water saturated conditions of the specimens, the volumetric changes expressed as percentages of their oven-dry volumes were·

Group A—29 per cent
Group C—35 per cent
Group E—37 per cent

The soil absorbed 22 per cent less moisture when the density of the soil was increased 40 lb. per cu. ft.; but, on the other hand, the fluctuation of densities in the soil was increased a total of 15 lb. per cu. ft. and the volumetric change of the soil from oven-dryness to saturation was increased 8 per cent.

CONCLUSIONS

These experiments have not yet been carried far enough in cycles of wetting and drying nor with enough different types of clay soil to support definite conclusions. Only two clay soils, SI and SII, have been investigated and the SI specimens have not yet been subjected to as many cycles of wetting and drying as the

SII specimens As far as the SI soil work has been carried, it checks in a general way with the SII soil data.

From the data thus far secured it appears in general that:

(1) The greater the density to which an expansive clay soil is compacted the greater will be the vertical movements in the riding surface when subsequent moisture content fluctuations occur in the soil.

(2) The soil substructure should not be made any denser than is necessary for the soil to carry the maximum loads transmitted thereto without appreciable movement

(3) Where the maximum load a pavement will transmit to a subgrade is known, practical laboratory tests can be made to determine the density to which a soil must be compacted to carry that load without moving appreciably when the soil absorbs its maximum moisture.

(4) There is a strong indication that soil substructures, therefore, should be compacted to different densities in accordance with the load supporting characteristics of the soils when wet and in accordance with the loads transmitted to the soils by the pavement or riding surface. In high embankments and substructures the weight of overlying soils, of course, must be taken into account.