LARGE-SCALE MODEL STUDIES OF HIGHWAY SUBDRAINAGE

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SYNOPSIS

Satisfactory methods for the design of subdrainage installations have yet to be developed. To help meet part of this need, a series of model tests was conducted to study the problem of lowering high ground-water by two parallel drains.

This report discusses some previous misconceptions concerning subdrainage and reviews the necessary theoretical background of the problem. It shows the desirability of comparing different drainage installations and the drainage characteristics of different soil types on the basis of the rate at which the ground-water surface is lowered. Of the various methods for the analysis of flow problems, the use of large-scale models was selected as being the best means for obtaining the desired information.

A wooden tank was constructed to simulate half of the symmetrical section under study. This tank was 6 ft. high, 4 ft. wide, and 22 ft long. Tests were conducted with drains varying from 4 to 8 in in diameter These were placed at depths of 1.5 to 4 5 ft., and at distances of 4 to 8 ft. from the centerline During these tests, measurements were taken of the discharge from the drain, the distribution of flow, and the position of the ground-water surface

A model law was derived which permitted extension of the results to any other scale and any other soil type, within certain limitations. Check tests with soils of widely different coefficients of permeability were conducted to verify this model law

On the basis of the test data and their extension by the model law, the effects of soil type, drain depth, drain spacing, and drain size upon the rate at which the ground-water is lowered and upon the rate of discharge into the drain are shown From this information several conclusions regarding proper drain location, depth, and size are drawn, and a method of estimating the drainability of soils is suggested.

PURPOSE AND SCOPE

The control of water in the soil under and near engineering structures continues to be a problem of foremost importance The detrimental effects of excessive soil moisture upon highway structures are well known to all it reduces the load-carrying capacity of the soil, it furnishes water for frost-heaving, and it contributes to slides and erosion

In most cases, it is possible and desirable to control moisture by surface drainage alone. There are instances, however, when it becomes necessary to install sub-surface drains to insure proper moisture conditions Such is the case when the ground-water surface is undesirably high or when sub-surface strata of pervious soils carry seepage water under the structure from somewhere at the side It is also desirable to provide drains for porous insulation courses under highway or airport pavements to prevent the possibility of these porous layers impounding water that might leak through cracks in the pavement

Much work has been done towards solving the critical problem of surface drainage A very comprehensive and easily applied method of design for surface drainage systems on airports, which is applicable to other problems as well, has been released by the Office of the Chief of Engineers, Washington (1) [1] However, no such method for the design of subdrains is available to aid the inexperienced designer. Information is available concerning methods of installing drains, and a few rule-of-the-thumb tables have been offered showing the spacing of drains in different soils (2) This does not, however, give the engineer a method for determining the proper location, depth, and size of subdrains for various problems. This study was directed therefore toward

[1] Numbers in parentheses refer to the list of references at the end of the paper

filling a part of that need, namely the design of subdrainage installations for lowering high ground-water surfaces under highways. The particular section assumed for study was one of level terrain with two symmetrically placed subdrains parallel to the roadway.

THE PROBLEM OF LOWERING A HIGH GROUND-WATER SURFACE

In designing a subdrainage system for the purpose of lowering a high ground-water surface there are several problems which confront the engineer. First he must determine whether or not the texture of the soil is such that subdrainage will be effective in a reasonable length of time. If he concludes that it is, then he must decide where to place the drains, how deep to put them, and what size to make them. He must also determine what type of subdrains

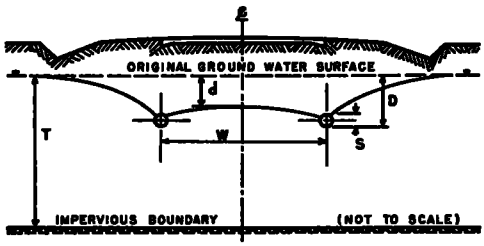

Figure 1. Typical Section Investigated in Model Studies

to use, and what filter of backfill material will be satisfactory for the soil encountered and the drain chosen.

Soil texture influences both the permeability and the capillary-moisture characteristics of a soil and through these two factors governs the drainability of that soil Permeability affects drainage action by limiting the rate at which free water can leave the soil and flow into the drain. The capillary-moisture characteristics of a soil also affect drainability by limiting the amount of water in the soil that can be removed by drainage. Therefore, in determining the advisability of attempting to drain a certain soil, both of these factors should be evaluated. There are some clear-cut cases, of course, in which one or the other alone will make it obvious that drainage is or is not practical. However, within the range of soil textures which are doubtful as to drainability, both permeability and capillarity should be considered.

In the author's opinion, the most workable concept for studying capillary-moisture is the "capillary-potential" or "energy" concept. This approach, in use for many years by the soil physicists, was first introduced to the engineering profession by Spangler and Russell in 1941 (3) This aspect of soil moisture was not included in this study, however, and only the action of free or gravitational water was under consideration. Results of these tests should therefore be taken only as a partial guide for determining drainability. The questions of drain type and backfill material were also omitted, but authoritative information is available regarding both of them (4)

The particular variables investigated in this study are those soil factors which influence the lowering of the ground-water surface and those dimensions which characterize any given installation, sometimes referred to as the boundary conditions. They have been designated as follows:

Soil factors,

- k—coefficient of permeability
- m—non-capillary porosity or the amount of drainable water per unit volume of soil. This value varies with the distance above the ground-water surface, and for these tests it was determined by measuring the non-capillary voids in the first 6 in of soil immediately above the ground-water surface.[2]

Dimensions defining boundary conditions,

- D—drain depth, measured from the original ground-water surface to the bottom of the drain
- S—drain diameter
- W—drain spacing or distance between drains
- T—thickness of the pervious layer under water.

The boundary conditions and symbols are diagrammatically shown in Figure 1. The time variables for any given section, by means of which the above variables are evaluated, are:

- t—time elapsed since the drain began removing water

[2] See Baver (5) for a discussion of the significance of non-capillary porosity, its variation with different soil types, and methods for its determination

d—depth of drainage at time t—the amount the ground-water surface has been lowered below its original position (measured at the centerline of the road unless otherwise stated)

q—rate of discharge per unit length of pipe, at time t.

EVALUATION OF DRAINAGE INSTALLATIONS ON A TIME BASIS

In order to approach the problem of subdrainage, it is first necessary to study the manner in which drains perform In Figure 2 is shown a series of views of the same typical cross-section. These views represent different time intervals after the drain starts removing water from the soil. Figure 2A represents the flow pattern that exists when the drains first start removing water from the soil. The lines representing the direction of flow were selected so that the same quantity of water flows in each "channel"—that is, between each pair of lines As indicated, there are eleven of these channels flowing into each drain. Figure 2B represents the flow pattern obtained a short while after the drains began removing water. The water table has been depressed below and to the side of the roadway. As the flow lines indicate, most of the water flowing into the drain is still derived from the water surface near the drain The number of channels flowing into each drain has been reduced to seven, and the discharge rate into the drain has decreased proportionately. In Figure 2C is shown the flow pattern which develops after a comparatively long time. The water surface in it has been lowered below its previous positions, and the water is flowing principally from the water surface far to the side of the road. The flow channels have decreased in number to two for each drain, and the rate of discharge has dropped an equivalent amount.

Taken from an actual drainage study, these sections show how drains progressively lower the water level under the roadway and depress the original water level at the side to progressively greater distances This progressive effect of the drain on the water surface is independent of the soil type. That is, assuming similar boundary conditions and stratifications if any, these successive stages will occur regardless of what soil is present What does depend on the soil type, of course, is the rate at which this effect takes place.

A false impression has been created in the minds of many engineers regarding the effects of soil texture upon the action of drains According to this misconception, the soil type determines the amount that a drain will alter the ground-water surface Supposedly, the more pervious the soil the lower this resulting position would be Actually, a given drainage installation will produce the same series of effects in any soil, with only the time required to produce a given effect being different for different soils.

It therefore seems logical to make comparisons between different installations, or between the drainage characteristics of different soils, on the basis of time Furthermore, since the purpose of the drains is to lower the ground-water level under the road, it seems desirable to determine in particular the effects

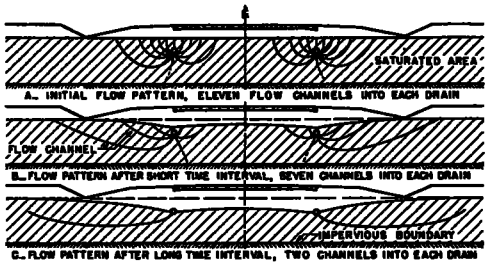

Figure 2. Successive Patterns of Flow Toward Parallel Drains; Flow Lines Shown Were Chosen so that the Same Rate of Discharge Flows in each Channel

of the different variables on the rate of increase in the depth of drainage under the centerline of road. The depth of drainage, d, as shown in Figure 1, is measured from the original position of the ground-water surface. It would be an added advantage to know the effect of these same variables on the rate of discharge per unit length of pipe, q, since a drain size must be selected for each installation.

METHODS OF INVESTIGATION OF FLOW PROBLEMS

In the investigation of flow problems, many methods are available If the boundary conditions are quite simple, mathematical solutions are sometimes possible (6). More adaptable to different problems and of easier application are the method of electrical models, based on the analogy between the flow of water and

the flow of electricity (7), (8), (9), and the graphical method utilizing flow-nets (10). None of these methods of solution, however, is adaptable to transient or unsteady-flow problems, and therefore cannot be used to evaluate different variables on a time basis as desired in this study. The use of actual soil-and-water models, therefore, becomes the only satisfactory approach to the problem. Small-scale models with one transparent wall have frequently been used for qualitative tests or for demonstration purposes (11) (12), but are generally considered unreliable from the standpoint of quantitative results. It was therefore determined to use large-scale models in this study.

LARGE-SCALE MODELS FOR DRAINAGE STUDIES

In planning a tank that would simulate the assumed cross-section in Figure 1, advantage was taken of symmetry and only half of the section reproduced. With this arrangement, as shown in Figure 3, one end of the tank represented the centerline of the roadway and will be referred to as such herein.

The tank was built of 2 in. by 8 in. tongued and grooved pine. It was made 4 ft. wide, 6 ft. deep, and 22 ft. long. A 2-ft. section at one end was partitioned off with a wire screen to serve as an inflow chamber. Overflow pipes were provided in this section at selected intervals to control the water elevation. A series of metal drains was made up, each being 4 ft. long, closed on the ends, and provided with a connection for a 2-in. hose at its bottom-center. The drains were 4, 6, and 8 in. in diameter and were perforated on the lower quarter. In each test, one of the drains was installed from 4 to 8 ft. from the centerline and at a depth of 1.5 to 4.5 ft. To permit changing the position of the drain in this manner, the discharge from the drain was carried out the bottom of the tank by means of a flexible coupling. Outside the tank, the discharge pipe was connected to a pipe-bent which had an open tee at its top. By rotating this bent, the open tee could be placed at the same elevation as the bottom of the drain for each test, assuring atmospheric pressure at the drain. A photograph of the tank is shown in Figure 4.

To provide data on discharge rates, the effluent pipe was carried to a small tank where the discharge rate could be determined either by weight or by volume, depending upon the magnitude of flow. In order to determine the distribution of flow and the position of the ground-water surface at any time, five groups of piezometer tubes were installed along the length of the tank at points indicated in Figure 3. In each group were nine tubes in

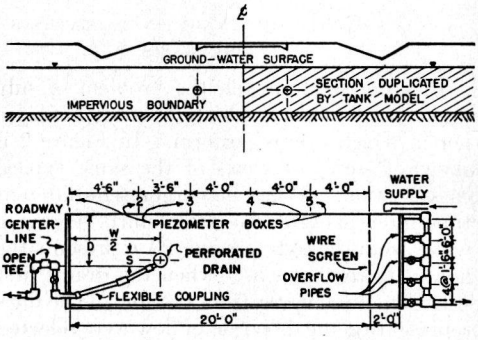

Figure 3. Details of Tank Model

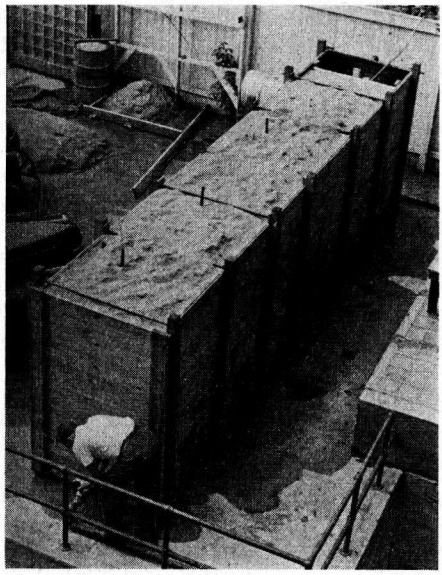

Figure 4. Tank Used in the Drainage Model Studies

contact with the soil in the tank through holes spaced at 6 in. intervals from 18 to 66 in. above the bottom of the tank. A scale placed alongside each group permitted reading the total head or potential at each of the nine points of contact. The heads were measured in inches above the bottom of the tank, the bottom being used as a datum plane.

Three soils were used in the tests—a coarse concrete sand, a silty sand, and a clean, fine sand. The tests with these soils were designated as Series A, B, and C, respectively. The grain-size distribution curves for the three soils are shown in Figure 5 The average soil properties which determine their drainage characteristics are:

Test Series	Soil Type	Coefficient of Permeability	Non-Capillary Porosity
A	Coarse Sand	3×10^{-3} ft per sec	0 12
B	Silty Sand	1×10^{-5} ft per sec	0 07
C	Fine Sand	1×10^{-4} ft per sec	0 10

Tests with the coarse sand showed that the flow was turbulent near the drain and that Darcy's law d d not hold. Tests with the silty sand were handicapped by experimental difficulties, and as a result only a small number of tests were completed. The fine sand used in Series C was intermediate between the other two, and its performance was satisfactory. The discussion of all the variables except soil type is therefore made on the basis of tests from Series C.

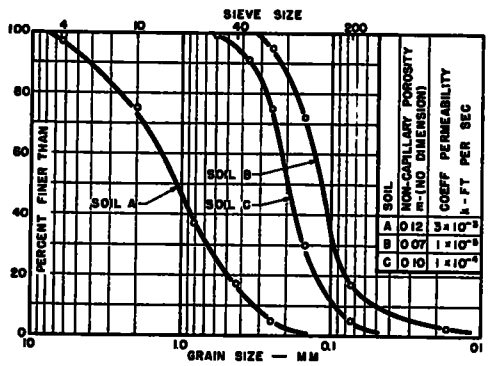

Figure 5. Soils Used in Model Studies

An effort was made to achieve homogeneity of the soil by placing it under water at all times In spite of this precaution there were some variations in permeability, but test data permitted the computation of the coefficient for each individual test and made possible corrections for those variations

In all of the tests except where otherwise noted, the water level was held constant at the inflow-end of the tank In a few tests, all inflow was stopped after the water level was established in the tank. After the start of each test, readings of the discharge rate and the piezometer tubes were taken at intervals until equilibrium was established and the readings became constant The time required for each test was from 1 to 20 hr, depending on the soil type. The piezometer tube readings at each box or group were all recorded simultaneously, by eye if the movement was slow, or by camera if the movement was rapid. A typical photographic reading is shown in Figure 6.

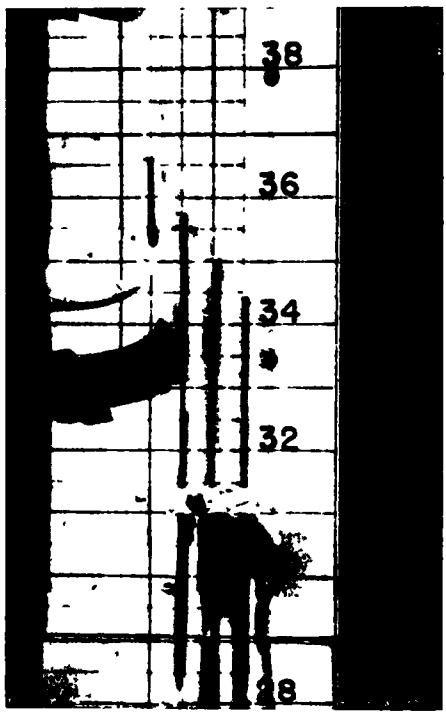

Figure 6. Typical Reading of Piezometer Tubes During a Test

DARCY'S LAW

In order to analyze and interpret the experimental data, it was necessary to take full advantage of the theory of flow through soils. The fundamental basis for all such theory is an empirical law published by H. M Darcy in 1856 and known as Darcy's Law. One of the usual ways of stating this law is that the rate of flow at any point in a flow system is directly proportional to the hydraulic gradient at that point This law may be expressed in

either of the following forms:

$$\left.\begin{aligned} v &= k\,i \\ \text{or} \quad Q &= k\,i\,A\,t \end{aligned}\right\} \qquad (1)$$

in which

v = discharge velocity
k = coefficient of permeability
i = hydraulic gradient
Q = total discharge
A = area of flow
t = discharge time.

This law may be explained with reference to Figure 7 A soil sample of length, l, is exposed to a head of water, h_1, at one end and a smaller head, h_2, at the other Due to this difference in potential, water will flow through the sample at a rate directly proportional to the hydraulic gradient,

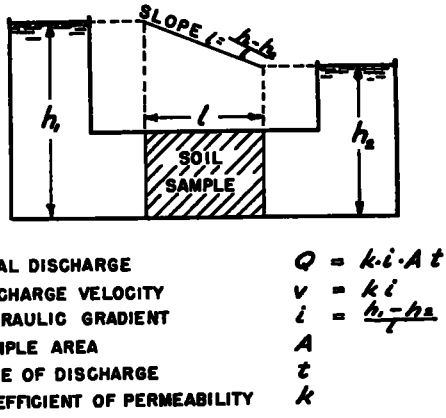

Figure 7. Darcy's Law

$$i = \frac{h_1 - h_2}{l}.$$

The constant of proportionality, k, is known as the coefficient of permeability. This coefficient is not a constant for any particular soil type as commonly supposed, but varies widely with the number, size, and shape of the soil pore spaces, the density and viscosity of the water flowing, and the chemical characteristics of both the soil and water It is a constant, therefore, only when all of these factors are constant. This points to the necessity of determining the coefficient of permeability with the soil in place and with natural water flowing, if anything more than qualitative results are desired.

It is natural that the validity of a law as fundamental as Darcy's should be the object of many more recent investigations Muskat (6) has made a thorough review of the several investigations and concludes that the law is valid providing Reynold's Number for the flow condition is less than one For computing the limiting value of Reynold's Number, d in the number is taken as a reasonable average of the grain diameters and v as the average velocity of the water. This limitation is a broad one and permits the application of Darcy's Law to any soil finer than a fairly coarse sand.

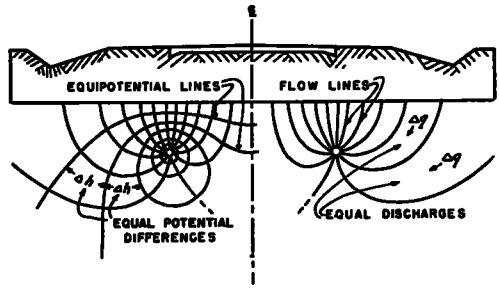

Figure 8. Flow Net Construction

THE FLOW-NET

In analyzing flow problems, a graphical tool known as the "flow-net" is frequently useful. In any section in which water is flowing there is a family of curved lines which represent the direction of flow In constructing a flow-net, a group of these lines is determined in such a manner that the same quantity of water flows between each pair of lines, as in the right half of Figure 8 There also exists another family of curves known as equipotential lines, each member of which connects all points of a certain potential. If the soil is isotropic, this family of curves will cross the family of flow lines at right angles In the construction of a flow-net, a group of these equipotential lines is chosen so that there is the same difference of potential between each pair. If this potential interval is chosen of such an amount that curvilinear squares are formed by the intersecting lines, a standard flow-net is formed, as in the left half of Figure 8.

As a direct consequence of Darcy's Law, the rate of discharge per unit length for any section where the flow is two-dimensional may be

determined from its flow-net by the formula:

$$q = kh \frac{N_f}{N_p} \qquad (2)$$

in which

q = rate of discharge per unit length
k = coefficient of permeability
h = total difference in head or potential
N_p = number of potential divisions
N_f = number of flow divisions.

Thus, if a flow-net can be constructed for a flow problem and either q or k is known, the other can be computed readily from this equation.

The construction of a flow-net requires a knowledge of all the boundary conditions or the experimental determination of one family of lines in the net. In the tests reported, flow-nets were constructed by plotting equipotential lines from experimental data and graphically constructing flow lines to complete the net. Using the flow-nets, the measured rate of discharge, and the equation above, the coefficient of permeability for each test was computed.

MODEL LAW FOR THE TESTS[3]

In model studies, it is necessary to be able to extend the results of tests to different scales, and if possible, to different materials In the derivation of the model law for these tests, the following notation will be used:

q = rate of discharge per unit length, cu. ft. per sec. per ft, or sq ft. per sec.
k = coefficient of permeability, ft per sec.
D = drain depth, used as a typical dimension, ft.
m = non-capillary porosity, that portion of a unit volume of soil occupied by drainable water, a dimensionless decimal fraction.
t = total elapsed time, sec
C = constant of proportionality, dimensionless

Subscript 1 refers to the model, and subscript 2 to the prototype.

Consider what happens during an elapsed time, Δt, in a section of infinite width in which the flow is two-dimensional and in which the soil is homogeneous. The ground-water surface drops from one position to a lower one as shown in Figure 9 The available water between the two positions of the ground-water is drawn out of the soil and, under the assumed conditions, is equal in volume to the water that flows into the drain during the same time interval. The volume, per unit depth of cross-section, drained out of the soil in the time, Δt, can be expressed as $C\,D^2\,m$. The constant C in this expression is a constant of proportionality relating the shaded area in Figure 9 to the area D^2. Any other typical dimension could be used in place of D, substituting an appropriate constant for C The following equations for the discharge, based on the volume of water removed from the soil, may then be written for model and prototype:

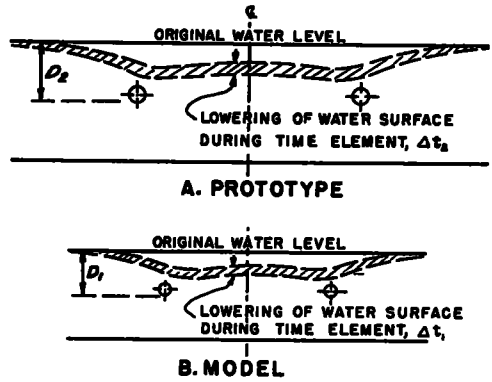

Figure 9. Geometrically Similar Conditions in Model and Prototype

$$\left.\begin{aligned} q_1\ \Delta t_1 &= C_1\,D_1^2\,m_1 \\ \text{and} \quad q_2\ \Delta t_2 &= C_2\,D_2^2\,m_2 \end{aligned}\right\} \qquad (3)$$

If corresponding (but not necessarily equal) times have elapsed in both model and prototype so that the ground-water surfaces as well as all other boundaries are geometrically similar, then C_1 equals C_2 and equations (3) can be rewritten:

$$\frac{q_1}{q_2} = \left(\frac{D_1}{D_2}\right)^2 \frac{m_1}{m_2}\frac{\Delta t_2}{\Delta t_1} \qquad (4)$$

From Darcy's Law applied to the flow-net, expressions for the rate of discharge into the drain in model and prototype can also be written:

$$\begin{aligned} q_1 &= k_1 D_1 \frac{N_{f1}}{N_{p1}} \\ q_2 &= k_2 D_2 \frac{N_{f2}}{N_{p2}} \end{aligned} \qquad (5)$$

[3] The derivation of this model law is gratefully taken, with some modifications, from an unprinted memorandum written especially for this study by Dr W E Howland, Professor of Sanitary Engineering, Purdue University

Again making use of the complete geometric similarity, it can be seen that

$\frac{N_{f1}}{N_{p1}}$ equals $\frac{N_{f2}}{N_{p2}}$ so that (5) can be rewritten

$$\frac{q_1}{q_2} = \frac{k_1 D_1}{k_2 D_2} \tag{6}$$

Using equations (4) and (6) and equating the rate of discharge into the drain to the rate of water removal from the soil, it follows that:

$$\frac{\Delta t_1}{\Delta t_2} = \frac{t_1}{t_2} = \frac{D_1 m_1 k_2}{D_2 m_2 k_1} \tag{7}$$

This equation is the model law for these tests and gives the relationship between the time intervals required to establish geometrically similar conditions in the model and its prototype It shows that the time required to establish any specified position of the ground-water surface varies inversely with the coefficient of permeability and directly with the non-capillary porosity and the model scale.

On the basis of this model law, results obtained from one model can be extended to geometrically similar cases of different scales and of different soil types, providing the necessary soil "constants" are known. As stated before, the section assumed for this derivation was infinite, a condition which cannot be duplicated either in the field or in model studies. As the following analysis of results shows, however, the variations from this assumption have but little effect on results when the variation occurs at a short distance away from the drain.

ANALYSIS OF RESULTS

As stated in the previous discussion, it was determined to investigate the different variables on the basis of discharge rates and the rate at which the ground water is lowered. For the following analysis, therefore, the depth-of-drainage data and the discharge data are plotted against various forms of time scales, comparing tests in which all but one variable were held constant.

Effect of Tank Limits

For purposes of the model tests, some assumption was necessary as to the source of water flowing through the soil Under actual conditions, the ground water which extends an infinite distance each side of the road is the source Since it was necessary to limit the length of the model tank, an artificial supply was provided by means of an inflow chamber at one end of the tank.

During most of the tests, the water level in the inflow chamber was held constant at some selected elevation. In general, the effect of this arrangement was to slow the process of drainage somewhat more than would be natural and to limit the amount the ground-water surface could be lowered. Thus, the ground-water surface, instead of approaching the level of the drain at infinite time, approached an intermediate position as its equilibrium.

A few tests were conducted in which no water was added after the start of the test. Under these conditions, the supply of water

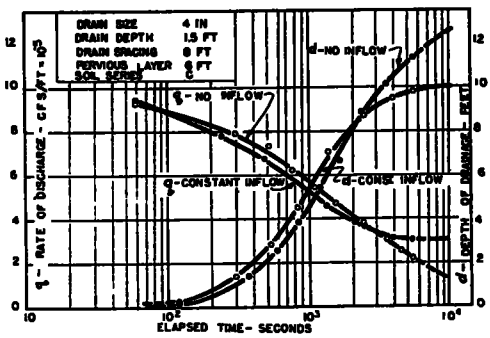

Figure 10. Effect of Tank Limits

was limited to that in the soil and in the open chamber at the end of the tank. Obviously, the water level in this case would be lowered somewhat faster than it would if there were an infinite body of soil to the side from which to draw water.

Thus, it was possible with different inflow arrangements to bracket the "ideal" condition of an infinite source even though it was impossible to obtain that condition itself. In Figure 10 are plotted data from two tests which were identical in every respect except that one had a constant inflow-level at the end of the tank and the other had no inflow. As the curves show, the results are almost identical for the first and most significant part of the tests and diverge only after an appreciable lapse of time Since the curve of the ideal condition would fall between the curves of these two tests, it may be concluded that

results from the first part of the tests may be considered independent of the tank limitations and independent of the manner of water-level control at the end of the tank.

Effect of Soil Type

If, instead of plotting drainage depth, d, and discharge rate, q, against time, t, as in Figure 10, the dimensionless factors d/D, $q/k\ D$, and $t\ k/m\ D$ are used respectively, the resulting curve should be independent of scale or soil type according to the model law developed, the only limitation being that Darcy's Law must hold for the flow system. Using these dimensionless factors, results from similar tests with the soils A, B, and C, previously described, are plotted in Figure 11. Computation of Reynold's Number for the flow

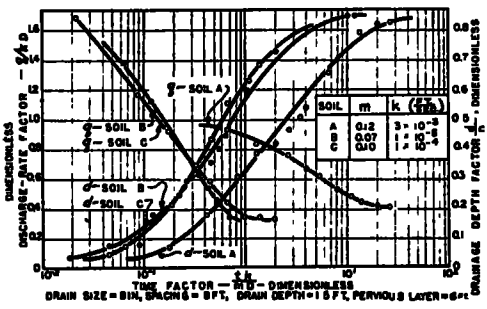

Figure 11. Effect of Soil Type

condition with Soil A showed that the critical value of one for that number was exceeded, with the result that Darcy's Law—and therefore the model law—did not apply. The Reynold's Numbers for the tests with Soils B and C were both below the limiting value and therefore Darcy's Law and the model law should apply. This relationship is verified by Figure 11. The results from the tests with Soils B and C are nearly identical, and the results from the test with Soil A indicate a slower rate of drainage resulting from turbulent flow. Thus, for soils in which the flow follows Darcy's Law, the effect of soil type upon the rate of drainage is given by the model law.

To show the application of this law, a plot of drainage-depth against time for one drainage installation is given in Figure 12. This installation is a 100 per cent enlargement of one of the model tests using Soil C with D equal to 1.5 ft., W equal to 12 ft., T equal to 6 ft., and S equal to 4 in. Three typical soils, designated, "X," "Y," and "Z," were assumed and time scales were computed for each using the model law. These scales show how greatly the time required to produce a given depth of drainage is affected by normal differences in permeability A series of such curves for typical installations would permit a designer to determine, on a time basis, how effectively drains could be expected to lower the ground-water surface in a particular soil. For instance, the time required to lower the water surface in some soils, such as Soil "Z" in Figure 12, would probably indicate the inadvisability of attempting to drain that soil.

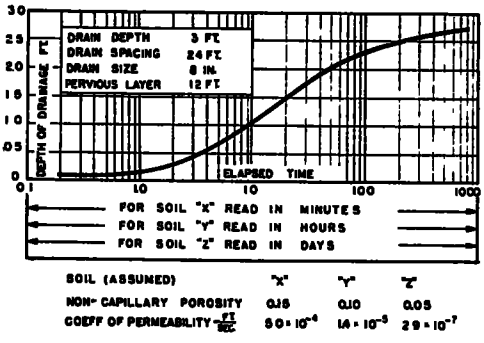

Figure 12. Application of Model Law

Effect of Drain Spacing

Tests were conducted with the same value of all variables except for drain spacing, W, which was varied in the model from 8 to 16 ft. The results are plotted in Figure 13. The tests were conducted with the same soil, but the results are plotted on dimensionless scales and are therefore independent of soil type and scale. The curves show the pronounced effect of drain spacing upon the rate at which the water between the drains is lowered. For the wider spacings, the time required to produce a certain depth of drainage is considerably longer than for the more narrow spacings. The fact that the initial rate of discharge is somewhat higher for the wider spacings is more than compensated for by the greater quantity of water between the two drains. The general conclusion can be drawn, therefore, that drains should be spaced as close to the pavement edges as other considerations permit

If they are placed out from the edge, they are less effective in removing water from under the pavement, and if they are placed much within the edge of the pavement the sharp rise in the ground-water surface on the outer side of the drain will keep the soil under the edge of the pavement saturated for a longer time.

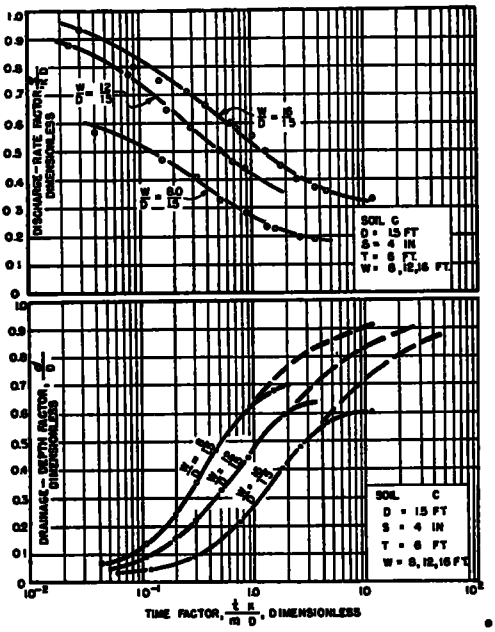

Figure 13. Effect of Drain Spacing

Effect of Drain Depth

To investigate the effect of drain depth, D, tests were performed in which all other variables were held constant while the drain depth was varied from 1 5 to 4 5 ft Results from these tests are plotted in Figure 14. The scales for these curves are plotted using T as the typical dimension rather than D as before, in order to show the effect of the variation in drain depth more clearly. The test for a drain depth of 4.5 ft. in this series was erratic, due to probable silting of the drain, and for the purposes of this report the curve for D equal to 4.5 ft. was interpolated from related data in another series.

Due to the higher heads on the lower drains, the rate of discharge for those installations was higher than for the drain at lesser depths. Consequently, for the range of values investigated, the ground-water surface at the centerline was lowered much faster by the deeper drains With regard to the effect produced, therefore, there is a decided advantage to placing drains deeper, not only because the maximum possible amount of drainage is higher but also because the initial rate of drainage is higher. There undoubtedly is a

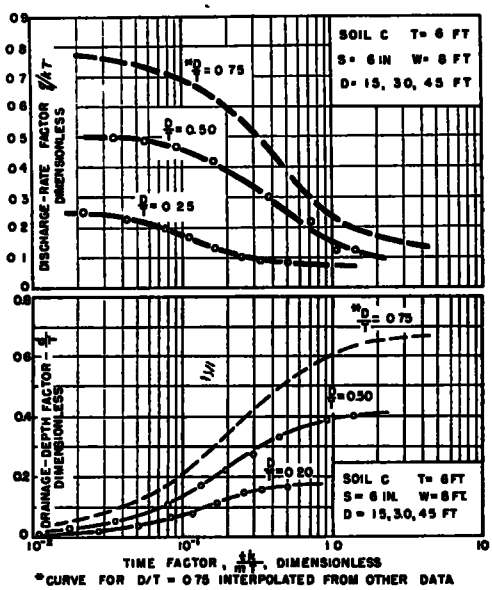

Figure 14. Effect of Drain Depth

limit to the depth at which an increase in D would produce beneficial results, probably in the order of D equal to W, the drain spacing. Economic and other limitations, however, would clearly restrict the drain depth to much lower values. Kirkham (11) found that the benefits of increased drain depth ended when the drain was placed three-quarters of the way down in the previous layer. This particular limitation would not hold, however, if the drains were backfilled with pervious material.

Effect of Drain Size

The size of a subdrain has two separate and distinct effects upon its ability to remove ground water. Not only does it limit the capacity of the drain to carry water, but it also limits the ability of the drain to admit water from the surrounding soil The latter effect is due to the notable influence of the

circumference of the drain upon the pattern of flow. In regard to this effect, the trench width should properly be considered as the drain diameter providing the trench is backfilled with a properly designed pervious material. However, if the backfill material does not function effectively as a filter, the true drain diameter controls. In these tests, no pervious backfill was used, so that the drain sizes quoted can be considered as such for similar installations or as trench widths when pervious backfill is used.

For these tests, the drain diameter was varied from 4 to 8 in. while the other variables

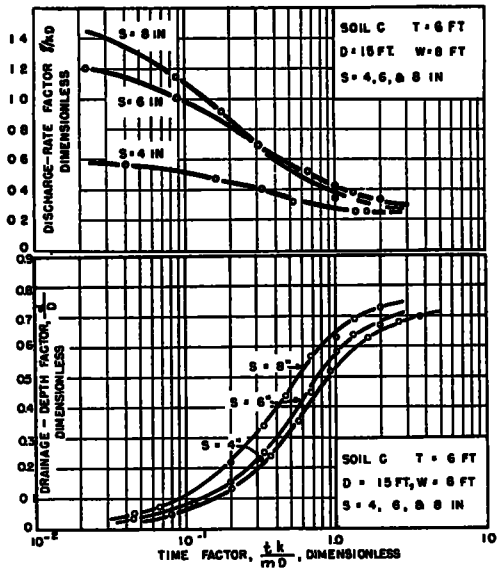

Figure 15. Effect of Drain Size

were held constant. The results are plotted in Figure 15. As would be expected, the larger sizes produce a higher rate of discharge as well as a higher rate of increase in depth of drainage. This effect, however, is obviously quite small in comparison to the effects of other variables.

Effect of Pervious-Layer Thickness

A series of tests was performed to investigate the effect of the thickness of pervious layer Holding the other variables constant, T was varied from 3 to 6 ft. by changing the water level in the tank. The results of these tests are shown in Figure 16. An interesting feature of the results is that the initial rate of discharge is appreciably larger for the greater thicknesses of pervious layer. The reason for this becomes apparent upon examination of the patterns of flow toward the drains. Much of the water flowing into the drain flows down below them before finally coming up and into the drains. The deeper pervious layers provide a greater area of flow beneath the drains and thus result in higher discharges into each drain. An apparent contradiction is shown by the slower rates of increase in drainage depth at the centerline when the pervious layer is deeper. It is supposed that the reason for this effect is that when T is greater and larger

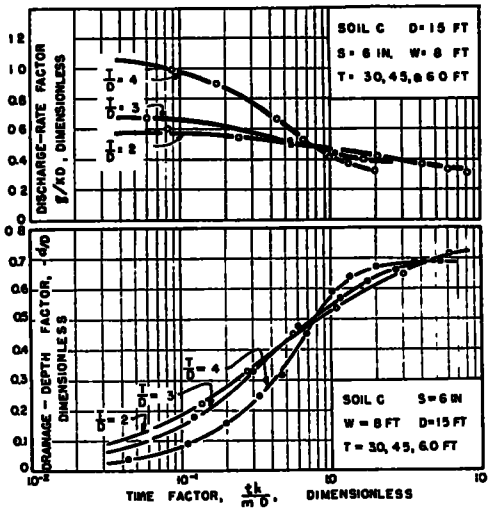

Figure 16. Effect of Pervious-Layer Thickness

quantities are flowing, more water from the outer side of the drains flows beneath the drain, reducing the area of flow for water coming from between the drains.

CONCLUSIONS

From the preceding analysis of results, the following observations and conclusions may be stated.

1 The effect of soil type upon rate of drainage is expressed adequately by the model law derived in this paper providing Darcy's Law holds According to the model law, the time required to produce a given depth of drainage varies inversely as the coefficient of permeability of the soil and directly as its noncapillary porosity.

2 If a curve is available providing the rate of lowering of the ground-water surface by a drainage installation geometrically similar to one being considered, the time required to produce various depths of drainage may be determined by application of the model law. These time intervals will provide a partial index to the drainability of the soil encountered using such an installation

3. With drainage installations of similar dimensions to those tested, Darcy's Law will apply safely to soils with coefficient of permeability 1 by 10^{-4} ft. per sec or less.

4 The limitation in the length of the tank used in the model studies produced little or no effect during the early and more significant part of the tests

5 From results of the tests investigating drain spacing, it can be stated that the most favorable location of longitudinal drains is near the edge of the pavement

6 Within the limits tested, drainage effectiveness increased as the depth to the drain was increased This effect should continue if the drain position was lowered to the bottom of the pervious layer providing pervious backfill were used When native backfill is used, the effectiveness should increase with increases in drain depth only up to three-fourths the pervious layer thickness, according to other investigators (11).

7. Within the range investigated, drain size had a small but noticeable effect upon the rate of drainage.

8. If pervious backfill is used, the drain size should be determined hydraulically—that is, to carry the required rate of discharge, this rate being determined by means of the pattern of flow into the drain, or, if available, from discharge curves for geometrically similar installations.

9 For equal values of other dimensions and increasing values for the thickness of pervious layer, the rate of discharge increases and the rate at which the ground-water level is lowered at the centerline decreases.

The writer wishes to acknowledge with appreciation the assistance of Professor K B. Woods, Assistant Director of the Joint Highway Research Project, Dr W E Howland, Professor of Sanitary Engineering, and Dr Ralph Fadum, Professor of Soil Mechanics, all of Purdue University, Dr. P. C. Rutledge, Northwestern University, formerly Professor of Soil Mechanics at Purdue University; and the many members of the staff of the Joint Highway Research Project who contributed to this work, in particular Mr. A. H. Layman, Research Engineer, who assisted with many of the tests, and Mr. W. H. Goetz, Research Engineer, who generously assisted in the preparation of this report. It is also desired to acknowledge Mr. G. W McAlpin, formerly Research Engineer for the Joint Highway Research Project who originated this study and whose reports are in the list of references.

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DISCUSSION ON HIGHWAY SUBDRAINAGE

DR W. E. HOWLAND, *Purdue University:* The model law for the flow of water through soil stated in various formulae by the author (as in 6 and 7) may be established in a variety of ways. As derived by the author the condition is assumed in both model and prototype that the rate of water coming to the drain is equal to the rate of depletion of water stored in the ground. This condition might not seem to the reader to be a very general one. For example, it may appear that added proof is needed to show that the law is also applicable to the author's experiments in which a constant level of water was maintained in a well at the end of the tank simulating an influx of water from a distant inexhaustible source as from a portion of the water table constantly replenished.

Extension of Applicability of the Model Law

Perhaps the easiest way to extend the proof given by the author to this somewhat more general case is merely to assume that the depletion of storage in the time Δt, namely, CD_m^2 (see Eq. 3) is not equal to the corresponding flow into the drain $q\Delta t$, but instead is some fraction F of that quantity. This fraction F must be the same in both model and prototype for similar patterns of flow. Then one could write in place of formulae 3, the following.

$$q_1\Delta t_1 = F\, CD_1^2 m_1$$

$$q_2\Delta t_2 = F\, CD_2^2 m_2$$

Then formulae 4 and the others dependent thereon would follow as well from the formulae just stated as from the author's formulae (3).

Alternative Derivation

The same model laws can be derived from a consideration of the constants which appear in the differential equation for unsteady two dimensional flow in sand.

Let q = the rate of flow past a typical section as, for example, Section a, a, a, a, y units high, and 1 unit wide of Fig. 17. (Its dimensions are those of area per unit time)

m = porosity, a dimensionless quantity, and

k = permeability defined by

$q = ky\dfrac{\partial y}{\partial x}$, the total differential of which with respect to distance is

$$q = \left(k\left(\frac{\partial y}{\partial x}\right)^2 + ky\frac{\partial^2 y}{\partial x^2}\right)\partial x$$

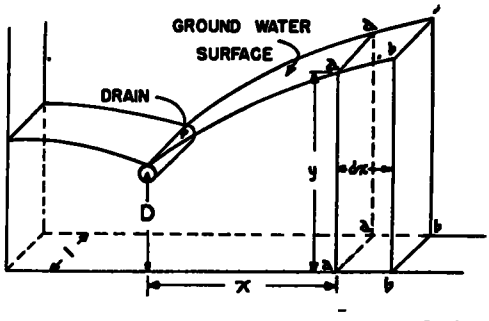

Figure 17. Unit Section of Drain and Soil

This is the difference in the rates of flow between two sections, a and b, δx distant from one another. The total amount of difference of flow in the time, δt, is this quantity multiplied by δt Now, in a section of the tank, δx long, and one unit wide, as the water level drops a distance of δy in time, δt, an amount of water would be drawn out of the sand equal to $m\delta x\delta y$.

Then $$\left(k\left(\frac{\partial y}{\partial x}\right)^2 + ky\frac{\partial^2 y}{\partial x^2}\right)\partial x\,\partial t = m\,\partial x\,\partial y$$

Or $$\left(\frac{\partial y}{\partial x}\right)^2 + y\frac{\partial^2 y}{\partial x^2} = \frac{m}{k}\frac{\partial y}{\partial t}$$

Now this equation will be modified to give it a truly dimensionless form. For this purpose the following substitutions will be made:

$$x_1 = \frac{x}{D} \quad \text{also} \quad \partial x_1 = \frac{\partial x}{D}$$

$$y_1 = \frac{y}{D} \quad \text{also} \quad \partial y_1 = \frac{\partial y}{D}$$

where D is a typical distance.

Also, $$t_1 = \frac{t}{t_a} \quad \text{and} \quad \partial t_1 = \frac{\partial t}{t_a}$$

where t_a is a typical time, as for example, the time for a midpoint on the water table to lower one-quarter of the total distance.

Then the equation becomes:

$$\left(\frac{\partial y_1}{\partial x_1}\right)^2 + y_1 \frac{\partial^2 y_1}{\partial x_1^2} = \left(\frac{mD}{kt_a}\right) \frac{\partial y_1}{\partial t_1}$$

All variables as y_1 and δy_1, etc are ratios and thus are truly dimensionless quantities, and the dimensionless constant which remains in the equation is $\frac{mD}{kt_a}$ or its inverse $\frac{kt_a}{mD}$. Thus, if $\frac{mD}{kt_a}$ is the same in one situation as in another, the same geometric patterns of flow will result in corresponding times From this principle all the model laws previously derived may be obtained.

Applicability of the Model Law

The model laws here considered are limited in their applicability It may be well to emphasize some of these limitations Clearly, if the model law is to hold, the same variations from a homogeneous and isotropic condition of the soil must exist in both model and prototype. Since variations are very hard to control it is probably correct to state that departures from this condition in the soil will result in apparent errors in tests of the model law when applied to two different soils. This, I believe to be the important reason for the discrepancies noted between soils B and C in the experiments shown on Figure 11.

Furthermore, the model law does not apply to turbulent flow nor to conditions in which changes in velocity head are significant in their relation to pressure head changes as the author has stated in reference to soil A, Figure 11. Another important difficulty arises from a difference in the amount of capillarity in two otherwise comparable soils. Although it may be possible to allow for this difference in certain cases, one should certainly be on his guard against too hasty an application of the model law to a pair of soils in one or both of which capillarity is an important factor, as the author has stated. For this reason it seems doubtful that the model law would apply as assumed to the hypothetical soil Z shown in Figure 12, whose capillarity is very high. The zone of capillary saturation would be large and could not be disregarded as an available flow channel. In fact, the drain might find itself in the zone of capillarity, in which case practically no water would be removed by the drain. Thus the application of the model law to this situation is beset with difficulties.

It should also be borne in mind that the object of drainage is to remove the water and not merely to lower the free water surface, which may be predicted by a model test. In fine-grained soils the height of capillary rise may be great. In these soils the lowering of the free water surface may not result in an appreciable reduction of moisture.

Viscous Fluid Flow Analogy to Two-dimensional Non-uniform Flow in Soils

If the model law just derived is applicable to soils, with limitations, (some of which have just been considered) then it ought to be possible to represent the flow in these soils by a properly devised model using not soil at all but a viscous fluid between correctly spaced parallel transparent plates The position of the drain adjacent to the model (but not thickness) should be geometrically similar to those of the prototype. This model law will now be derived

Using the same methods as is given in standard texts to derive the Hagen-Poiseuille equation for viscous flow in circular pipes (see Fluid Mechanics by Dodge and Thompson p. 171), it is possible to show that the actual mean velocity of flow between parallel plates is $\frac{b_1^2}{12} \frac{g}{\nu_1} i$. Here i, is the pressure head gradient, b_1 is the distance between the plates, g is the acceleration of gravity and ν_1 is the kinematic viscosity of the viscous fluid. Now the law of flow of water through the sand on which the model laws just considered have been

based is that V_2 mean $= k_2 i_2$. Thus $\frac{b_1^2}{12}\frac{g}{\nu_1}$ takes the place in the author's formula No. 7 of k_1. This is a sort of equivalent permeability and may replace k_1 wherever this k_1 appears in previously derived model laws. Also, it is to be noted that m_1, the porosity in this model, will now be unity. The author's formula (7) may then be written as

$$\frac{t_1}{t_2} = \frac{D_1 k_2 12 \nu_1}{D_2 b_1^2 g m_2}$$

and (6) becomes

$$\frac{q_1}{q_2} = \frac{D_1 \quad b_1^2 g}{k_2 D_2 \nu_1 12}$$

Warning: These values of q are rates in volume per unit time per unit length of drain or in dimensions of area divided by time

It is hoped that such a model may be employed in future studies. It seems to offer many advantages over other methods for determining the effect of drains on lowering ground water levels for two dimensional stream line flow through soils of known characteristics in which capillarity does not play an important role.

PROF. W. J. SCHLICK, *Engineering Experiment Station, Iowa State College:* Mr McClelland, and the Joint Highway Research Project, Purdue University, merit commendation for an interesting report of a carefully made study. The consideration of the time-factor, and the use of the model law in arriving at practical applications of the data, are particularly interesting

I was impressed particularly by the similarity in methods, and the differences in results, for this Purdue study and one which our Station reported to the First International Congress of Soil Science [1] We also tried three soils; with two of these we experienced "experimental difficulties," though some results, not included in the published paper, were obtained for soil No 2 Soil No. 1 was similar to Soil "C", Soil No 1 was slightly coarser than Soil C, and had values of $k = 1.5 \times 10^{-4}$ at 55° F, and of $m = 0\ 14$ [2] Soil No. 2 was a finer, "less permeable loam," with a value of $m = 0.08$.

It is unfortunate that this earlier study was not reported in a source more readily available to soil engineers. It is not now feasible to present more than a brief resume of it.

The principal differences in results appear to be due to differences in dimensional relationships. Our model (Fig 1) was smaller, and used a smaller drain, though it provided (in effect) a larger spacing-depth ratio, and a larger ratio of depth of pervious layer below the drain to depth of drainable soil

The depths of soil above and below the drain were 8-in. and 7-in., respectively, and were

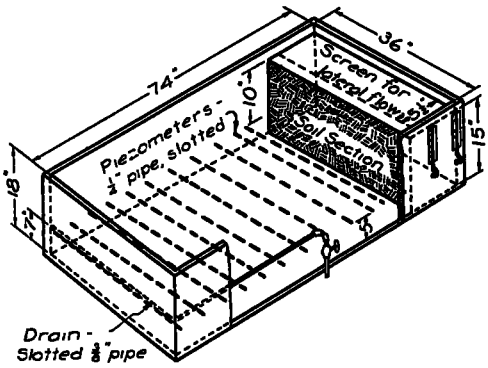

Figure 1

constant for all trials. The drain was used either in the middle or at one end of the model.

Our drain was a ⅜-in. brass pipe, slotted transversely every ½-in. both top and bottom; it was surrounded with sand so that its outside diameter was about 1¼ in. The Purdue drains were 4, 6 and 8 in in diameter This difference in diameter should influence the shape and the slope of the ground-water curves near the drain. As the size of the drain increases there will be an increase in the minimum cross-section of soil through which the flow must pass Field records include many instances where the water table at the drain was above the drain; this probably was due to some combination of entrance head into the drain, resistance to flow through the minimum cross-section of soil, and possibly surcharged flow in the drain. Some trials with our model showed equal heads at equal (short) distances laterally and below the drain

The values of k in our studies were determined as illustrated by Figure 2, using a

[1] *Proceedings*, International Congress of Soil Science, Vol 4-5, 686-700 (1927)

[2] The difference between the "pore space on basis of true specific gravity, per cent" and "Water retained after draining, by volume per cent."

range of losses in head The values for soil No. 1 were checked later by certain data obtained with the model

Our results showed certain distinct differences from those reported by Mr. McClelland. We found, (1) a straight-line relationship between temperature and the permeability coefficient; (2) for soil No. 1, different types of flow, and different head-discharge relationships, for what were termed "drainage flow"

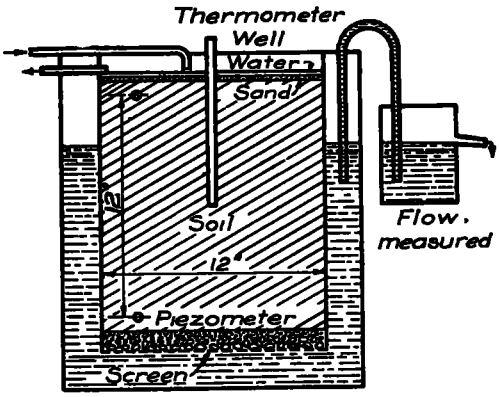

Figure 2

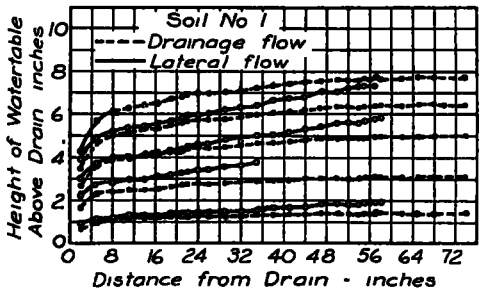

Figure 3

(flow from a saturated soil) and "lateral flow" (flow when water entered the soil from a compartment at one end (or both ends) of the model); (3) different head-discharge relationships for drainage flow through soils No. 1 and No. 2, and (4) a decrease in the permeability of soil No 2 from day to day

The results of determinations of k for soil No 1 showed that the discharge varied 1 54 per cent of that at 50 F for each degree of temperature variation from 50 F, this relationship was used in correcting all rates of discharge for the model. The temperature variation for different trials of a series usually was only 1 or 2 deg., but the range between individual trials of several series was as much as 10 deg.

The results for lateral flow through soil No. 1 show the groundwater curves to be (nearly) straight lines to within 8 or 10 in. from the drain. When these data are plotted to a condensed time-scale (Figure 3), the "straight" portions are found to be very flat curves. All of the flow enters the soil column at one end; since each succeeding cross-section is smaller, a slightly steeper slope is required to maintain the flow. It seems possible that the propor-

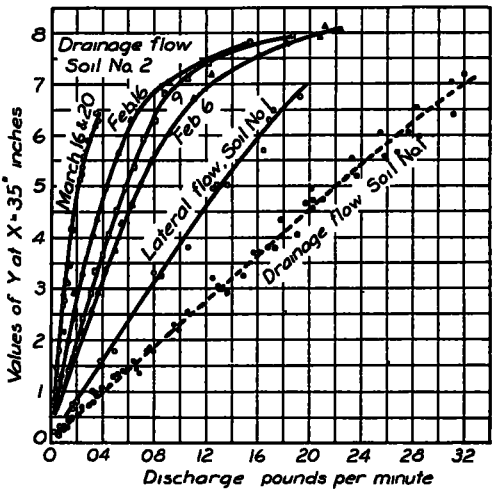

Figure 4

tionately-great depth of the pervious layer beneath the drain so decreased the reduction in cross-section that it tended to obscure the resulting increase in slope This in turn suggests that the curvature of the watertable is some function of the relative capacity of the pervious layer below the drain.

For drainage flow water enters the saturated zone at all points along the length of the column, the resulting increase in the quantity of flow requires an increase in slope or head (Fig 3) The rates of discharge for a given value of "head" are greater than for lateral flow, as illustrated by Figure 4.

The results for soil No. 2 were not so consistent as those for soil No 1, but they indicate some differences in head-discharge relation-

ships for drainage flow (Fig 4). The fact that the upper ends of the graphs for soil No. 1 appear to have a slight curvature suggests that this difference may be partly one of degree.

The data for lateral flow through soil No. 2 (Fig. 5) illustrate three interesting points: (1) an appreciable entrance head; this phenomenon was observed also in determination of k for both soils No. 1 and No. 2; (2) a decrease in permeability from day to day; this is shown also by Figure 4; and (3) some flow passed in the drain, through the lower pervious stratum, when flow was from one side only; this may account for the fact that the average flow, at each head, from both sides was 2.7 times that from one side only.

The differences in the findings of these two studies appear to be due to differences in dimensions and dimensional relationships in the two models However, until these differences are explained, or at least till the limits within which they are important are determined, it seems unwise to accept the results of either for general application

Field Studies The action of drains, and the characteristics of the flow to them, under relatively impervious roadway surfaces have been subjects of discussion for many years. This Station undertook studies of this problem at two locations in Northern Iowa in 1921–24. Arrangements were made with the Iowa Highway Commission to have the drains installed at three positions—under the roadside ditches, 2 ft out from the edges of the pavement, and at intermediate locations, and to provide for vertical gage pipes through the pavement. The installations were made during the Summer and Fall of 1921 It was unfortunate, for this study, that a sandy or gravelly stratum was encountered at about the level of the drains in each case, and that the Spring and Summer seasons of 1922–24 were relatively "dry" Readings in the Fall of 1921 and during the Spring and Summer of the succeeding years showed the watertable under the pavement to be at, or only slightly above, the drains; these data indicated that the drains were performing their function, but furnished only negative indications as to the flow conditions under the pavement and between the drains

Although the results of these field studies were influenced by the special conditions, the absence of a normal groundwater curve between the drains, under the pavement, appears logical. Figure 1 of Mr. McClelland's report suggests this, though it is probable that in many cases the distances D and d will be more nearly equal. It seems desirable that any adopted design method should be based upon the characteristics of the flow from the sides and not upon the flow for a normal watertable curve between drains.

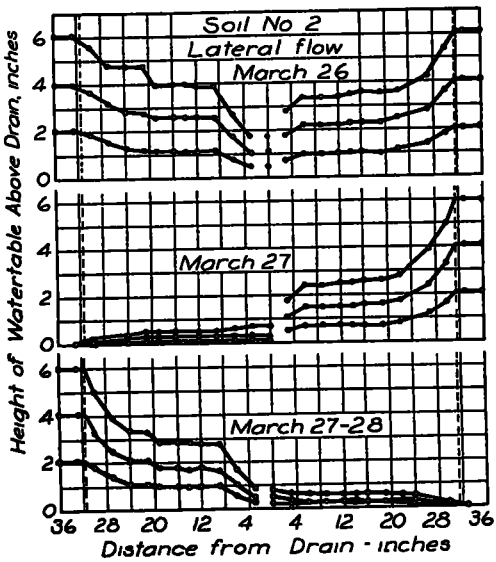

Figure 5

MR. MCCLELLAND: The model law derived by Dr Howland for applying the results of viscous-liquid model studies to two-dimensional problems of flow through soils opens the way to a method of investigation which is well worth consideration There is no doubt that once the mechanical details of model construction are accomplished, the technique should permit investigation of a much wider range of variables than is possible with more unwieldy methods It also eliminates some of the defects which hamper small soil-and-water models, such as the distorting effect of disproportionate capillary rise It is to be hoped that future investigators of this or similar problems will consider viscous-liquid models as a possible method of study

I do not agree with Dr Howland that the capillary characteristics of a soil can limit in

any way the application of the experimental results and the model law which have been presented. The only way in which the soil capillaries influence the movement of gravitational water is through their effect upon the "non-capillary porosity" of the soil, which factor is included in the model law as presented in the paper. I do agree, however, that it should be emphasized that any conclusions drawn from an application of these results will definitely be affected by considera-

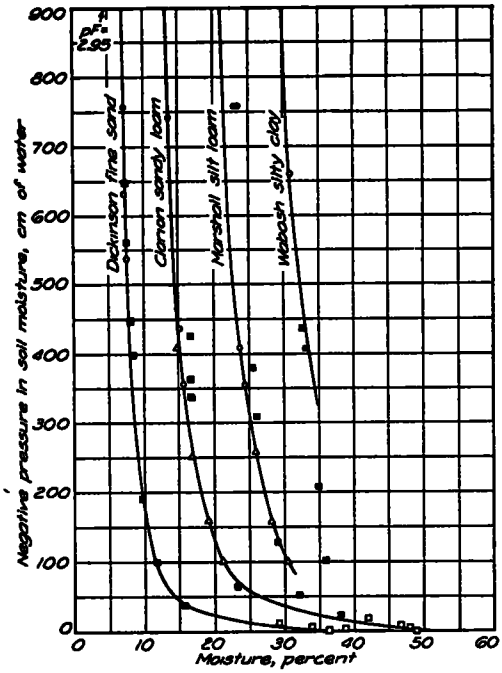

Figure 1

tion of the capillary characteristic of the soil in question

Thus, while model results and the model law may be validly applied with reasonable accuracy to the movement of ground water, a satisfactory analysis should be carried further. The purpose of drainage, of course, is to increase the supporting power of soil by reducing its moisture content Therefore, knowing the movement of ground water to be expected in a certain case, it is also necessary to determine what changes in moisture content in the soil will be caused by such a movement This can only be done by investigating the capillary characteristics of the particular soil in question

The best tool for this purpose is the energy concept of soil moisture, introduced to the engineering field by Spangler and Russell in 1940 (3). For any soil a continuous relationship exists between moisture content and the energy with which that moisture is held in the soil. This relationship may be determined experimentally, and results of such tests on four Iowa soils by Spangler and Russell are given in Figure 1. They also show that when moisture conditions are at equilibrium, the negative pressure with which the moisture is held in the soil is equivalent to the distance from that point to the free water surface. Assuming equilibrium conditions therefore, a reasonable assumption where both surface infiltration and evaporation are restricted by a paved surface, the curves shown in that figure may be interpreted as the variation in moisture content with the distance above the ground water surface.

To show how such information would apply to a drainage problem, assume that a pavement base is constructed on the "Dickinson fine sand" of Figure 1 Assume also that the ground-water surface is one foot below the base, and that the drainage installation given in Figure 12 is employed Using the time scale for Soil "Y" which corresponds very nearly to the Dickinson fine sand, it can be seen from Figure 12 that the drains will lower the ground-water surface an additional 2 5 ft in 200 hours From Figure 1 it can be seen that this would result in reduction in moisture content at the pavement base from 18 to about 11 per cent, which would probably represent a substantial increase in supporting power. Drainage in such a situation would without a doubt be successful

Assume a similar situation with the Marshall silt loam, which corresponds roughly to Soil "Z". In this instance, it would take 200 days to lower the ground-water surface an additional 2 5 ft Even at the end of this time, as shown by the moisture sorption curve, the surface moisture content would be reduced only from 35 per cent to 30 per cent. The effectiveness of drainage in such an instance might well be questioned.

Thus the capillary characteristics of soils

should not be considered as "limiting" the application of the model law, but rather as deserving joint consideration with such applications. A true design procedure for a drainage problem, although admittedly ideal for most applications, might include the following steps:

(1) Establish the supporting power to be required of the soil.

(2) Determine from strength tests what maximum moisture content in the soil will permit satisfaction of that requirement.

(3) Determine from a "moisture sorption curve" for the soil how deep the groundwater surface must be so that the proper moisture content at the surface will not be exceeded under equilibrium moisture conditions.

(4) Determine from an application of the model law and these or other model tests what installation and what length of time will be necessary to satisfy requirement (3).

Either of the last two steps may clearly indicate that drainage is not economically feasible and that some other method of correction should be applied.

WEIGHT-IN-WATER METHODS OF DETERMINING THE MOISTURE CONTENT OF SOIL-CEMENT MIXTURES IN THE FIELD

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SYNOPSIS

During the last two years soil-cement airport facilities have been constructed at rates of 10,000, 20,000 and more square yards of 6-in pavement per day. At this speed the control of moisture content for such a large volume of material becomes a very important item requiring careful planning and efficient testing technique Field moisture content determinations are generally made by drying a representative sample over a kerosene or gasoline burner, a system which has proved adequate in most instances. However, since this method is relatively slow, a continued search is being made for more rapid methods of determining these moisture contents This report discusses two "Weight-In-Water" methods which serve this purpose

The principle of these methods has been applied to moisture and specific gravity tests of concrete aggregates, but its application to soil-cement is relatively new. The methods are based on the fundamental that a sample of soil-cement when weighed in water weighs the same regardless of its moisture content After the weights of the sample in air and in water are obtained, the specific gravity of the mixture is used to aid in the calculation of its dry weight and moisture content.

The fact that cement is a powerful flocculating agent is instrumental in making possible the use of a pycnometer-syphon weight-in-water method applicable to soil-cement mixtures even though they are composed of fine textured soils.

During the past two years a number of soil-cement projects have been built at rates of 10,000 to 20,000 and more sq. yd of six-inch pavement a day. Much of this yardage was built using the "train lane" processing method, the fundamentals of which are shown in Figure 1. In this method, processing equipment completes the construction of each lane in a relatively short time and, therefore, rapid moisture determinations are required after the dry mix in order to control subsequent moisture application. Some additional moisture content determinations may be required during water application. Final moisture content checks are needed during final rolling so that a large number of moisture tests are required each day. As an illustration, if one moisture sample is taken for every 350 sq. yd. of