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HIGHWAY RESEARCH RECORD

Number 203

Symposium on Subsurface Drainage

7 Reports

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Subsurface Drainage

7 Reports

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Dedication

Mr. Edward S. Barber, 51, former Chairman of the Highway Research Board Committee on Sub-Surface Drainage, died suddenly on January 31, 1966. Ed, as he was known to all, was one of the doers as well as being one of the thinkers. His contributions to the field of subsurface drainage and soil mechanics are noted for their practicalities. He was a moving force and a constant contributor to the activities of the Sub-Surface Drainage Committee from its conception.

Mr. Barber was graduated with first honors from the University of Maryland in 1935 and did graduate work at George Washington University and Catholic University of America. He accepted a position with the U. S. Bureau of Public Roads in 1936 to work in the newly developing field of Soil Mechanics and Foundations. He joined the faculty at the University of Maryland in 1947 and was an Associate Professor of Civil Engineering there at the time of his death.

Mr. Barber continued his association with the Bureau of Public Roads as a Consultant on Soil Mechanics and Foundations while on the faculty at the University of Maryland. While serving in this broad capacity, he became well known as an expert in his field. He also entered private consulting work, and in 1957 while on leave from the University, acted as a Consultant to the Guatemalan Ministry of Communications and Public Works.

He was an active member of engineering societies, including the National Society of Professional Engineers, the American Society of Civil Engineers, the American Society for Engineering Education, and the Highway Research Board.

In addition to his membership on the Committee on Sub-Surface Drainage since 1945 and chairmanship from 1957 to 1963, Mr. Barber was a member of the Department of Soils, Geology and Foundations from 1940 until his death and served on or chaired eleven other committees and subcommittees in three departments of the Board.

During his association with the Highway Research Board, Mr. Barber made numerous contributions to highway engineering literature and particularly to the literature in his chosen field of work. Among these are two technical reports, 14 papers, and 11 discussions.

With fond recollections of enlightening and encouraging associations, this Symposium is dedicated to "Ed."

HRB Committee SGF-B6 on
Sub-Surface Drainage

Foreword

Although the fundamentals of seepage and drainage have been well developed and understood in academic circles for several decades, there has been mounting evidence that these principles are not consistently being applied. Recognizing the need for continued diligent efforts to get fundamental information on seepage and drainage into the hands of "users" and "doers," Sub-Surface Drainage Committee SGF-B6 was asked to sponsor a symposium for the January 1967 Highway Research Board's Annual Meeting. In honor of the memory of the late Edward S. Barber, former chairman of this committee, the symposium was designated the "Symposium on Subsurface Drainage–Memorial to E. S. Barber." This RECORD contains the five papers presented formally at the memorial symposium (the first five), together with two additional papers sponsored by other committees, but of such content as to appropriately be included in this publication.

This volume will be of value not only to materials engineers and research engineers having an interest in permeability testing techniques, but also to design, construction, and maintenance men as well. Soils engineers and geologists will find considerable worthwhile material in this collection of papers by eminent authors and notable research organizations.

From granulometric principles, Winterkorn demonstrates that desirable strength and permeability characteristics can be expected of mineral aggregates of relatively large dimensions, and of a single size or a very narrow range of sizes. His work emphasizes the importance of requiring graded filters with an internal core of highly permeable aggregate when a high capacity for conduction of groundwater and seepage is of primary concern.

Strohm, Nettles, and Calhoun describe and give results of work by the U. S. Army Engineer Waterways Experiment Station in developing a reliable, reproducible compaction procedure for testing drainage aggregates for permeability over a wide range of densities. Their work, which was done with a gyratory compactor apparatus, proves that under very high compactive efforts, drainage aggregates which contain as little as 5 percent of fines become virtually useless as conductors of seepage. Furthermore, these materials can cause pavement distress through the buildup of excess pore pressures under traffic, and from frost action.

Pillsbury's report on experiments with filter materials for subdrains presents practical criteria for protecting drainpipes from soil intrusion. It points out, for example, that glass fiber filters may be highly compressed and lose much of their original permeability. He suggests that a glass fiber material with very coarse strands of borosilicate glass and an insoluble binder might be a suitable material to place around drain tiles, but the common insulation type is not.

Nettles and Schomaker have an interesting paper on Waterways Experiment Station research on the infiltration of soil through pipe openings. While this is not an unexplored field, these new tests provide criteria relating soil type and placement with susceptibility to washing through joints in pipes of various sizes under a range of water head, joint configuration, and externally induced vibration.

Parker and Jenne, in a well-illustrated paper, demonstrate that the process of subsurface erosion by soil piping is a much more common and serious phenomenon than might have been thought. The dominant type of piping which is damaging highway structures in the western drylands is pointed out to be that of the desiccation, or stress-desiccation crack type.

Although widely found in montmorillonitic valley alluvium and argillaceous bedrock, it also occurs in loess and other materials. Serious structural damage by piping usually can be prevented if susceptible materials are avoided whenever possible, and measures are taken to prevent concentrations of runoff when these materials cannot be avoided.

Galvin and Hanrahan describe full-scale field drainage experiments to determine the effect of drainage depth on the water table drawdown curve and the in-place permeability of peat soils. They compare actual drawdowns with those calculated by several theoretical methods. Their work showed that for the best drainage of blanket peat an optimum drainage depth exists. For their experiments this depth was found to be 3 ½ feet.

Mickle and Spangler compared the long-time accumulation of moisture in the soil subgrade beneath an impervious surface with the estimated equilibrium moisture content based on measurements of the moisture retention characteristics of the soil and the evaluation of the groundwater. A theoretical approach, based on thermodynamics, was used in evaluating the free energy per unit mass of water in a soil water system. They conclude that equilibrium moisture contents in soil columns under impervious surfaces can be predicted. They found that temperature has only a minor effect on ultimate moisture contents.

Harry R. Cedergren,
Chairman, Sub-surface Drainage
Committee SGF-B6

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Application of Granulometric Principles for Optimization of Strength and Permeability of Granular Drainage Structures

HANS F. WINTERKORN, Department of Civil and Geological Engineering, Princeton University

● GRANULOMETRY is the science of the physical properties of assemblies or systems composed of grain-like particles, such as sand, gravel, crushed rock, bird shot, seeds, and others. It embraces the study and measuring of the size, shape and surface features of individual particles as well as the influence of these properties and the gradation of the particles on the packing characteristics, mechanical resistance properties and permeability of multi-particle systems. Whereas granulometry is concerned mainly with these materials, many of its laws are of a geometric nature and are equally true for particles of atomic size and that of large boulders. This permits the utilization, for sand and gravel systems, of knowledge that was originally obtained on assemblies of atoms or molecules and vice versa.

Systems falling in the realm of granulometry are important elements of many highway drainage structures. In this role, their permeability or capacity to conduct water is of primary concern. It is equally important that this capacity be maintained. Therefore, the mechanical resistance properties of the system and of its particulate components must be that, under the static and dynamic service loads and the existing environmental conditions, the system will be able to function satisfactorily for the lifetime of the structure it is supposed to serve. Sources of partial or complete failure may be:

1. Shear failure or densification of the system without breakage or wear of the component particles, i.e., without change in granulometric composition.
2. Breakage and attrition of component particles with resulting change in granulometry and decrease in pore volume with concomitant increase in amount of internal surface per unit volume.
3. The infiltration of soil fines which have the same effect as described for the fines produced by breakage and attrition.

The second source may be eliminated by specifying adequate levels of strength and abrasion resistance of the aggregate determined by suitable test methods. The third source may be eliminated or minimized by the use of properly designed filter structures (9). The remaining problem is how to optimize shear resistance and permeability. The knowledge required to do this has been available for some time, but because it has been developed in different branches of engineering and material science, we tend to keep it in different compartments of our brain. It is the purpose of this paper to bring the pertinent facts and concepts together in a simple presentation.

FLOW OF WATER IN A GRANULAR SYSTEM

In drainage systems, our primary concern is the rate of flow of water. This rate is determined by the hydraulic gradient, by such system factors as total porosity and sizes and shapes of the pores or voids, and by the extent to which the pore or void channels are filled with water. Whatever the exact nature of the flow, the rules for optimization of the system are the same. This will be shown for the two extreme cases of Darcy

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Paper sponsored by Committee on Sub-Surface Drainage and presented of the 46th Annual Meeting.

type or laminar flow in saturated sand or gravel beds, and of open-channel type flow that may occur at partial filling of a coarse broken stone type of bed.

A system showing Darcy type of flow, characterized by the equation

$$dQ/dt = A\,k\,i \tag{1}$$

where

dQ/dt	=	volume rate of flow per unit time,
k	=	transmission coefficient,
i	=	hydraulic gradient, and
A	=	cross section,

may be replaced by a system composed of m parallel capillaries of radius r per unit cross section. Then we obtain according to the Hagen-Poisseuille equation

$$dQ/dt = \frac{A\,m\,r^4\,\pi}{8\,\eta} \tag{2}$$

where η is the viscosity of the water.

This means that the transmission constant k in the Darcy equation should correspond to the expression $m\,r^4\,\pi/8\,\eta$, or if we introduce a correction factor C to take care of the different geometry of the pores in the particulate system, we can write

$$k = \frac{C\,m\,r^4\,\pi}{8\,\eta}$$

Since $r^2\pi$ is the cross section of one capillary, $m\,r^2\pi$ is a two-dimensional expression of the porosity n of the capillary system and we obtain

$$k = \frac{C\,n\,r^2}{8\,\eta} \tag{3}$$

For a particular temperature, the viscosity η is a constant and we can include it in a new constant $C' = C/2\eta$. We further consider $r^2/4$ as a reduction of $(r^2\pi/2\pi)^2$, i.e., as the square of the ratio of capillary cross section over wetted perimeter. Multiplication of both the numerator and denominator of this ratio with a unit length and with the number of capillaries m per unit volume yields the ratio of porosity n over internal surface s per unit volume of the system. Hence,

$$k = C'\,n\,(n/s)^2 \tag{4}$$

Accordingly, the transmission coefficient or permeability of such a system is directly proportional to the third power of the porosity and inversely to the second power of solid surface per unit volume of the system. The surface per unit volume of solid is for spherical particles 3/r and for cubes 6/d where r and d are the sphere radius and the edge length of the cube respectively. Whether or not the previously derived equations hold rigorously, the unquestionable consequences for optimization of permeability are that: (a) the porosity of the system should be as large as possible, and (b) the size of the pores and hence of component particles should be as large as possible. We must check, of course, the compatibility of these conclusions with the required mechanical strength properties of the system.

With respect to the open-channel type of flow in a partially filled system of coarse rock fragments, we come to the same conclusion regarding flow optimization by inspecting the Chezy formulas:

$$V = c\sqrt{RS} \tag{5a}$$

$$Q = AV \tag{5b}$$

$$Q = A\,c\,\sqrt{RS} \tag{5c}$$

where
- Q = discharge in cu ft/sec,
- A = cross-sectional area of flow in sq ft,
- c = a roughness coefficient,
- R = mean hydraulic radius in ft = area of section/wetted perimeter, and
- S = slope or grade in ft per ft.

Checking the Laminar Flow Eq. 4

A good chance for checking Eq. 4 was offered by a fine set of data on grain-size composition, capillary rise and experimentally determined coefficients of permeability of nine granular materials published in 1959 (12). The measured capillary rise values permitted calculation of the ratio 2 $r\pi / r^2\pi$ or s/n from the formula

$$h = 2\,r\pi \cos\alpha\, T / r^2\pi\, g\,\gamma \tag{6}$$

where
- h = height of capillary rise,
- r = radius of capillary,
- γ = density of water,
- g = acceleration of gravity,
- T = surface tension of the liquid, and
- cos α = cosine of the angle of wetting between liquid and solid surface which for soil-water systems approaches unity.

The surface-area data were calculated from the given average grain sizes and the porosity data from the n/s and s data. The experimentally determined data and those derived therefrom by calculation are given in Table 1. Columns (f) and (g) permit comparison of the actual and the calculated permeability data. Despite the rather circuitous manner in which the calculations had to be made, the agreement is very close.

TABLE 1
INTERRELATIONSHIPS BETWEEN GRAIN SIZE, CAPILLARY RISE, INTERNAL SURFACE, POROSITY, AND HYDRAULIC PERMEABILITY OF GRANULAR SOIL SYSTEMS[1]

(a) Grain Sizes			(b)	(c)	(d)	(e)	(f) Coefficient of Hydraulic Permeability k (10^{-4} cm/sec)	(g)
Passing Sieve No.	Retained Sieve No.	Avg. Diameter (cm)	Saturated Capillary Rise (cm)	Surface per cm^3 Solid (cm^2)	Surface per cm^3 Pore Vol. From Cap. Rise Data	Porosity Calculated From (c) and (d)	Experimental	Calculated From Data in Columns (d) and (e)
10	20	0.118	6.4	51	85.4	37.5	50	23
20	30	0.069	9.4	81	125	39.3	23	13
30	40	0.049	13.2	122	176	41.0	13	6.6
40	60	0.031	20.0	194	267	41.0	6.7	2.7
60	80	0.021	29.8	290	397	42.3	5.7	1.3
80	100	0.016	35.6	370	475	43.8	2.7	0.7
100	140	0.012	47.0	488	628	43.8	1.6	0.55
140	200	0.0087	67.0	690	894	43.5	0.7	0.27
200	270	0.0062	90.5	970	1210	44.4	0.3	0.15

[1]Experimental data for grain size, capillary rise and permeability from Ref. (12).

PACKING CHARACTERISTICS AND BEARING CAPACITY OF NON-COHESIVE GRANULAR SYSTEMS

The bearing capacity of such systems may be defined as their ability to resist single and repeated loadings without rupture or excessive deformation, including volume change by internal readjustment or consolidation. All failure in loading involves the shear resistance either directly or indirectly. For cohesionless systems, this shear resistance is due to the internal friction and its magnitude can be expressed in accordance with Coulomb by:

$$\tau = \sigma_n \tan \varphi \tag{7}$$

where

τ = shear stress,
σ_n = the normal stress on the shear plane, and
$\tan \varphi$ = the coefficient of internal friction.

For our purpose, the problem of mechanical stability is to find the relationship between tan φ and various characteristics of the particles composing the system and of the system itself. Among the more important parameters are: size, shape, surface roughness, specific gravity, and size distribution of the granular components, and the packing characteristics and void ratios of the systems. Their influence shall be discussed on the basis of accumulated practical experience and data from dependable laboratory experimentation, supplemented whenever indicated by theoretical considerations.

Critical evaluation of a large number of test data has shown that tan φ for most natural sands can be expressed by:

$$\tan \varphi = \frac{C}{e - b} \tag{8}$$

where C is a constant which depends on interparticle friction and gradation, e is the void ratio, and b is another constant of the nature of a minimum void ratio. Both C and b are also pressure dependent at least in the case of laboratory tests where the sample size imposes certain limitations. This equation shows that tan φ increases with decreasing void ratios if the other parameters are kept constant (2, 8, 10).

Influence of Particle Size

If the other parameters are kept constant, the size of the component particles does not affect tan φ. Contrary data in the literature originate from the fact that with change in size of test material, even of glass ballotini, there is usually a change in surface characteristics–roughness, thickness and character of adsorbed surface films, etc.; in natural gravels and sands there is also usually a change in shape of the particles with change in size. Even with apparently perfectly spherical glass ballotini of the same chemical composition, the smaller sized particles cannot be packed in laboratory tests to the same density as the larger sized particles. Hence, there is often a packing effect that is falsely interpreted as a size effect (5, 7).

Influence of Specific Gravity

Marked differences in specific gravity of particles usually correspond to marked differences in chemical composition and surface properties of particles which again affect the void ratios that can be obtained by maximum and minimum packing effort. This is illustrated by the following data (3):

Material	Sp. Gr.	e (loose)	e (compacted)	Δ e
Lead shot	11.91	0.68	0.59	0.09
Sulfur shot	2.024	0.77	0.60	0.17
Dune sand	2.681	0.70	0.60	0.10
Theoretical packing		0.91 (cubic)	0.35 (rhombih.)	0.56

Accordingly, an increase in specific gravity tends to narrow the practically obtainable range in void ratios. However, there is no intrinsic effect of specific gravity on tan φ for otherwise equal particle and system conditions.

Influence of Particle Shape and Surface Roughness

There exist a number of different methods of measuring and expressing particle shape. If one actually determines by macro- or microscopic methods the length, width and thickness of particles, the best way of presentation is to give the closest geometric shape together with the ratios of the respective parameters. One may also use the volumetric coefficient (vc), which is the ratio of the actual volume of a particle to that of a sphere in which it can just be enclosed. For general engineering purposes, these methods are too time-consuming, and one arrives at practical shape or roundness facts from the porosity or void ratios obtained on granular materials placed in a container by one or another standardized method that results in reproducible, relatively loose packings or high void ratios. The greater the deviation from sphericity, the larger is the porosity obtained in such loose packing.

It can be reasoned that granular systems, composed of particles of the same size range and mineral matter but varying in shape, when densified by the same moderate standardized method are in comparable states as far as shear strength is concerned; and it can be concluded that the greater the porosity thus determined the larger is the constant C in Eq. 8. In other words, the tan φ for a certain material and void ratio increases with increasing deviation of the component particles from spherical shape. This can be true, even for the same mineral material, only if the deviation from spherical shape is relatively small as in the case of cubes (vc = 0.37) and angular stones (vc = 0.22), but certainly not for plate and needle shapes that may in random orientation form a very open framework similar to a house of cards. Therefore, to conclude from the high porosity of a crushed mica sample [determined by Graton and Fraser (3) as 93.53 percent in the loose and 86.62 percent in a compacted state] to a high internal friction would be a mistake no engineer should make. Incidentally, such cardhouse and related structures formed by particles of extreme shape are very sensitive to shock, vibrations and similar dynamic loadings, and are likely to densify to relatively small porosities if subjected to their action. As an extreme example of the porosities that can be reached by platelike materials, we may take that of a Na-bentonite sample for which Hogentogler (6) reported a shrinkage limit of 6 percent. Taking the specific gravity of water as 1 and that of the bentonite as 2.5, we obtain a porosity of 13 percent and a void ratio of 0.15. The more extreme the shape of the particles, the greater is the danger of stress concentration and development of bending stresses. Therefore, in practice such shapes can be admitted even to a limited degree only for minerals of very

TABLE 2

CHARACTERISTICS OF STONE SAND SAMPLES MILLED TO THREE DIFFERENT SHAPES

Parent Rock: Limestone of Bulk Dry Specific Gravity 2.71
Size Fraction: Sieve No. 16 - No. 30

Characteristic	Sample Designation		
	Rounded	Ground	Screening
Percent passing sieve No. 16	99.3	99.2	98.7
No. 30	4.0	2.5	2.5
Percent voids NCSA method	47.3	50.9	52.3
Void ratio e	0.9	1.03	1.1
tan φ at these void ratios	0.82	0.84	0.86
e (crit) from shear tests	0.93	1.0	1.06

Note: The samples, compositional data and voids values were supplied by J. E. Gray, Engineering Director of the National Crushed Stone Association. The shear tests were performed by Jean Dutertre, with the machine described by Herbst and Winterkorn (1, 5), The NCSA method is described by Gray ad Bell (4).

high strength, toughness and abrasion resistance. The effect of surface roughness parallels that of particle shape. The important practical problem is: to what extent will a favorable roughness be maintained under service conditions ?

The effect of shape on the angle of friction of granular assemblies is shown in Figure 1, which presents plots of tan φ vs void ratios of three limestone sands derived from the same rock but milled to different shapes. The designations and properties of these samples are given in Table 2. The data in Table 2 and Figure 1 show the following:

1. Deviation from spherical particle shape increases the tan φ of samples of the same mineral material and the same size fraction if densified to the same void ratio, and also increases the critical void ratios;

2. The void ratios obtained from the three samples by means of the NCSA densification method correspond quite closely with their respective critical void ratios obtained from the shear tests; and

3. At the void ratios obtained by the NCSA method, the samples are at comparable states of frictional resistance and hence of workability which is its inverse.

Systems Composed of Particles of Different Sizes

Systems of 2, 3 and more components of different sizes up to so-called continuous gradation, as employed in concrete technology and soil stabilization, have been investigated theoretically and experimentally by many workers in many different countries. The purpose of most of this work was to find combinations that would give the lowest

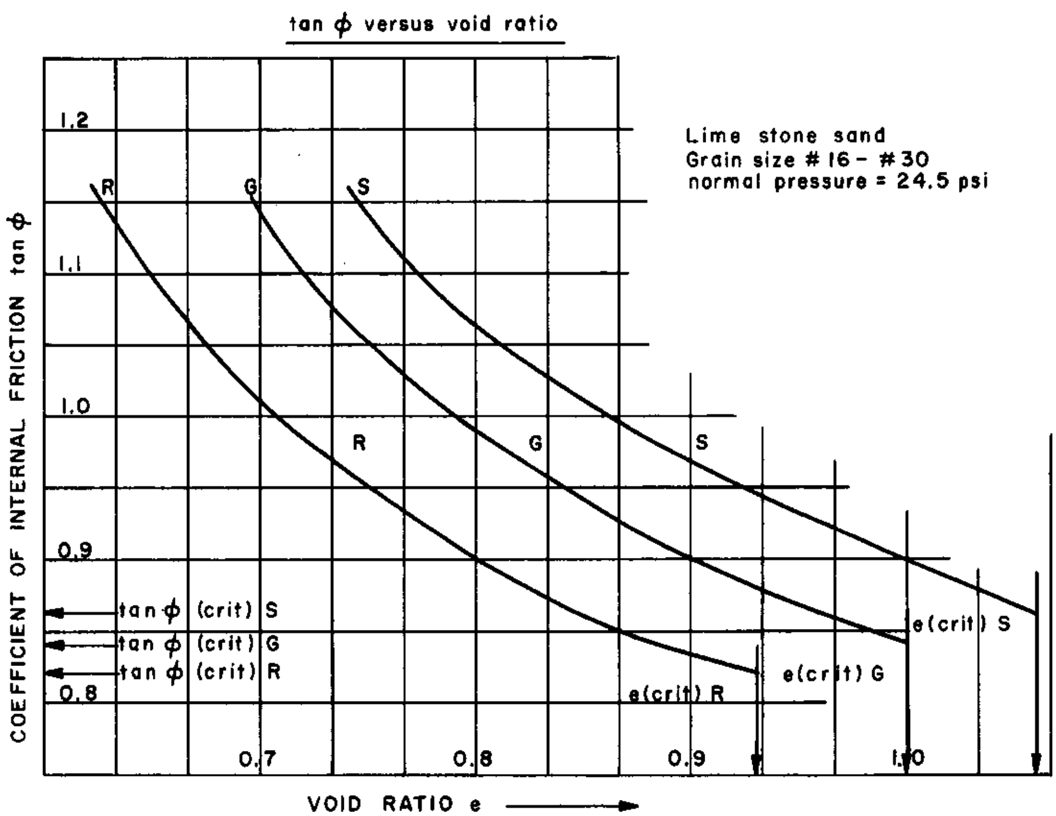

Figure 1. Relationship between tan φ and void ratio for 3 limestone sands of same size range but of different grain shapes.

percentages of voids with a given moderate densification procedure as in rodding sand and coarse aggregate mixtures. For all such binary, ternary, and multicomponent systems, combinations have been found that with the particular densification method employed give minimum percentages of voids; i.e., the greater the difference between the maximum and minimum size of the particulate components, the lower the minimum. But as long as the system was of non-cohesive granular character, i.e., the smallest size component not so minute or of such water affinity that capillary condensation and development of cohesion could occur, the angle of friction of the mixtures and hence the shear resistance and its inverse, the workability, were found to be essentially the same (5, 11). The obvious lesson for granular drainage structures is the use of single-sized coarse aggregate of maximum feasible dimensions. Of course, they will have to be protected externally by suitable filter layers.

CONCLUSION

Evaluation of available practical and theoretical information on the permeability and strength properties of granular systems makes it clear that these properties can be optimized by using the largest feasible aggregate of single size or very narrow size range. The aggregate should be of sufficient strength, toughness, abrasion resistance and durability to retain its integrity under service conditions, and its packing should be such that no further densification will occur.

ACKNOWLEDGMENT

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REFERENCES

1. Dutertre, J. C. Shear Strength of Natural Granular Materials. Masters Thesis, Princeton University, August 1966. Also, with Winterkorn, Hans F., Princeton Soil Engineering Research Series No. 6, 1966.
2. Farouki, O. T., and Winterkorn, Hans F. Mechanical Properties of Granular Systems. Highway Research Record 52, pp. 10-42, 1964.
3. Graton, L. C., and Fraser, N. J. Systematic Packing of Spheres with Particular Relation to Porosity and Permeability. Jour. of Geology, Vol. 43, pp. 785-909, 1935.
4. Gray, J. E., and Bell, J. E. Stone Sand. Eng. Bull. No. 13, NCSA, 1964.
5. Herbst, T. F., and Winterkorn, Hans F. Shear Phenomena in Granular Random Packings. Scientific Rept. No. 2, AFCRL-65-370, Princeton Univ., 1965.
6. Hogentogler, C. A. Engineering Properties of Soil. McGraw-Hill Book Co., New York, 1937.
7. Idel, Karl-Heinz. The Shear Strength of Granular Soil Materials. Bull. 2, Institute for Soil Mechanics and Earth Structures, Technical University of Karlsruhe, Germany, 1960 (in German).
8. Kezdi, Arpad. Discussion of Paper by Farouki and Winterkorn. Highway Research Record 52, pp. 42-58, 1964.
9. Subsurface Drainage. Engineering Manual, Cp. XXI. Design of Drainage Facilities for Airfields, Part II. U. S. War Department, Office of the Chief of Engineers, Washington, 1943.
10. Winterkorn, Hans F. Macromeritic Liquids. ASTM Spec. Tech. Publ. 156, pp. 77-89, 1953.
11. Winterkorn, Hans. F. Shear Resistance and Equation of State for Non-Cohesive Macromeritic Systems. Symposium on Equation of the State of the Earth, AGU, April 1966. Abstract in AGU Transactions, Vol. 47, No. 1, p. 175, 1966. Also, Princeton Soil Engineering Research Series No. 7, AFCRL-67-0048, Dept. of Civil and Geological Eng., Princeton Univ., 1966.
12. Subsurface Drainage of Highways and Airports. HRB Bull. 209, 1959.

Study of Drainage Characteristics of Base Course Materials

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This paper describes a laboratory investigation of the effect of density on the drainage characteristics (i.e., permeability and effective porosity) of base-course materials. Four gradations of base material compacted to various densities ranging between 90 and 110 percent of their respective CE-55 (modified) maximum dry density were studied. The investigation included the development of test procedures, the performance of laboratory tests, and analysis of data.

In developing adequate test procedures, the effects of the following factors on permeability test results were investigated: specimen preparation, hydraulic gradient, rearrangement of particles, and temperature of permeating water. The results of constant-head laboratory tests, using procedures developed in this study, show that the drainage characteristics of coarse-grained materials can be determined with sufficient accuracy to provide adequate design data. However, probable field conditions, such as hydraulic gradient and water temperature, must be taken into consideration in performing laboratory permeability-density tests.

● THE need for subsurface drainage has been recognized for many years, but it has been only within the last 50 years that design concepts have been developed to meet design needs. Theoretical concepts for base-course drainage were developed by Casagrande and Shannon (1) based on a subsurface drainage study conducted by the Corps of Engineers (CE) from 1945 to 1947 (2). Methods developed by Terzaghi (3) for design of filter courses around subdrains, modified by later investigations by the CE (4, 5, 6), and the theory developed by Casagrande and Wilson are the bases for the subsurface drainage design criteria contained in Department of the Army TM 5-820-2 (7).

The CE criteria for base-course drainage require subsurface drainage: (a) where frost action occurs in the subgrade or base course beneath the pavements; (b) where groundwater rises to the bottom of the base course; or (c) where the pavement may become inundated and there is little possibility of water draining from the base into the subgrade. Base drainage is sometimes required at the low point of longitudinal grades. To simplify the analyses of drainage of base courses, it is assumed that: (a) the base course to be drained is initially fully saturated; (b) no inflow occurs during drainage; (c) the subgrade constitutes an impervious boundary; and (d) the base course has a free outflow into the drainage trench.

The design requirements are that the drainage system be adequate to handle the maximum seepage flow and drain the base course so that a degree of drainage of 50 percent is obtained in not more than 10 days. Degree of drainage is defined as the ratio, expressed in percent, of the amount of water drained to the total amount of water that can drain by gravity from the material; this is dependent on the effective porosity and permeability of the material. Consequently, the principal factors considered in

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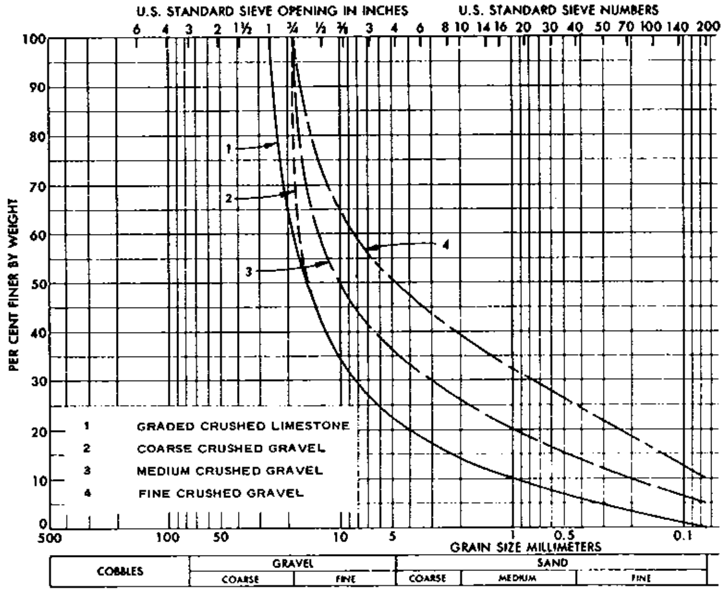


Figure 1. Gradation curves, gradations 1, 2, 3, and 4.

base-course drainage design are the geometry of the pavement and drainage system, and the coefficient of permeability and effective porosity of the base-course material.

The criteria in TM 5-820-2 for design of pavement base-course drainage facilities have been used since World War II. To handle the ever increasing wheel loads of the new aircraft, greater densities and thicknesses of base courses were required, and the allowable percentage of fines (material passing the No. 200 sieve) in the base-course material was reduced to facilitate drainage.

However, recent observations indicate that base courses and filter courses are periodically saturated at certain airfields, and some extensive subdrain systems appear to be largely ineffective. The need to improve design criteria adapted to present practices for constructing subdrain systems is recognized.

All aspects of subsurface drainage design criteria and related pavement design requirements were reviewed to investigate the causes of inadequate performance of subdrains. Based on this review, it is probable that drainage characteristics of the highly compacted, dense base courses required by present airfield pavements vary from those assumed in present subsurface design criteria. This led to the present study to investigate the effect of density on the drainage characteristics of base-course materials.

The purpose of the investigation is to develop better data on the drainage characteristics of highly compacted base-course materials for use in design of subdrain systems (8). It includes studies to develop test procedures, and the performance of laboratory tests to determine the permeability and effective porosity of four gradations of base-course materials.

SELECTION OF MATERIALS

The four gradations of base-course materials were selected to meet CE guide specifications for graded crushed aggregate (9) and stabilized aggregate (10) for base-course materials. Each specification includes three gradation bands, the principal difference between bands being the maximum particle size allowed. Gradations within the No. 3 (finest) band from each specification were selected to represent the most critical material with respect to drainage characteristics.

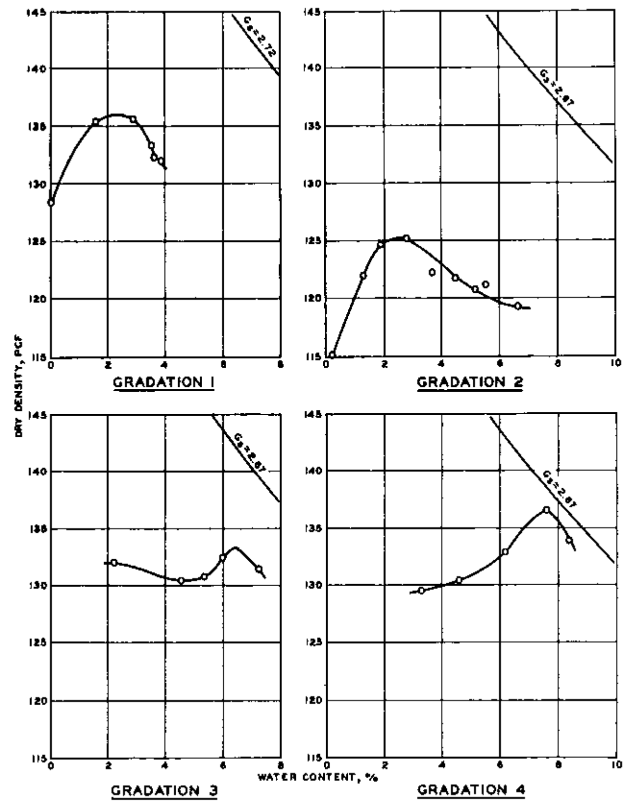


Figure 2. Compaction curves, gradations 1, 2, 3, and 4.

The desired gradations were obtained by combining various amounts of materials from aggregate storage bins. A subangular crushed limestone was blended to duplicate the coarse limit specified for graded crushed aggregate. Angular crushed gravel was used to reproduce the coarse, medium, and fine gradations within the limits specified for stabilized aggregate. The materials described are referred to as gradations 1, 2, 3, and 4 in the remainder of this paper. Gradation curves are shown in Figure 1.

Physical Properties

Compaction tests (impact type) were performed on the four gradations using procedures described in Military Standard 621A (11), except that material larger than ¾ in. in diameter was not removed as specified in the procedures. To determine a maximum dry density for reference, the CE-55 compaction effort was used and is equivalent to the modified AASHO effort. The compaction curves are shown in Figure 2. Pertinent physical properties of the materials are given in Table 1.

PROCEDURES FOR PREPARING PERMEABILITY TEST SPECIMENS

During initial testing, constant head permeability tests were performed on impact compacted specimens of gradation 1. Results for different densities and water heads were inconsistent, and attempts to reproduce results were unsuccessful. A limited investigation was made of compaction methods to determine factors contributing to the inconsistencies in permeability results. The effects of specimen size, molding water content, and specimen saturation also were considered.

TABLE 1

SUMMARY OF PHYSICAL CHARACTERISTICS

Gradation	Material	Classification[a]	Particle Shape	Sp. Gr. of Solids, G_s	Max. Particle Size (in.)	Percent Passing No. 200 Sieve	D_{10} Size (mm)	Atterberg Limits of Binder[b]	Max. CE-55 Dry Density (lb/cu ft)	Opt. CE-55 Water Content (% dry wt)
1	Limestone	GP	Subangular	2.72	1	0	0.94	NP	136.0	2.3
2	Chert	GP	Angular	2.67	¾	0	0.94	NP	125.5	2.5
3	Chert	GW-GM	Angular	2.67	¾	5	0.20	NP	133.3	6.4
4	Chert	GW-GM	Angular	2.67	¾	10	0.074	NP	136.5	7.5

[a]Unified Soil Classification System.
[b]Material passing No. 40 sieve.

Compaction Methods

In addition to the impact compaction method, the vibratory table and gyratory compaction methods were studied to determine the most suitable way of obtaining uniform specimens for the range in densities desired.

Several undesirable characteristics were noted in the impact method. It was difficult to strike off the top of a specimen and obtain a uniform height for density measurements. Grain-size distribution curves obtained on the materials before and after compaction indicated degradation which increased with increase in the compaction effort. Stratification occurred from compacting the material in layers. Greater stratification occurred at the higher compaction efforts because greater degradation occurred on the surface of each compacted layer. It was also difficult to compact two or more specimens to identical densities for the purpose of checking or reproducing results.

Two specimens, one 6 in. and one 12 in. in diameter, were prepared by vibratory compaction (12), and examination of these specimens after permeability testing indicated that, although degradation was eliminated, excessive segregation of particle sizes resulted. In addition, it was not possible to obtain the range of densities desired for the permeability study. Consequently, the vibratory table method of compaction was considered unsuitable.

Gyratory compacted specimens were prepared using the gyratory apparatus shown in Figure 3; a schematic section through the gyrating mechanism is shown in Figure 4. The gyratory compactor (13, 14) produces a controlled, uniform kneading action of the specimen through a gyratory motion of the mold, while a constant static pressure is maintained at each end of the specimen by two steel rams whose faces always remain parallel. The compaction effort applied to the specimen can be controlled through three variables of the compactor: (a) static pressure up to 250 psi applied to the ends of the specimen; (b) angle of gyration of the roller assemblies variable from 0 to 2 deg; and (c) number of revolutions of gyration, unlimited.

The specimens were compacted in 6-in. diameter gyratory molds to a height of approximately 5 in. The undesirable characteristics of the specimens compacted by the impact or the vibratory-table methods were either eliminated or reduced to within tolerable limits in the specimens compacted by the gyratory method. There was little or no segregation of particles or stratification of the specimen. Grain-size distribution curves obtained on specimens before and after compaction to the same density by the impact and gyratory methods are shown in Figure 5. They indicate that while there was some degradation of the larger particles by the gyratory compaction method, it was considerably less than that produced by the impact method. No trimming of the ends of the specimen was required since the faces of the two steel rams are flat and remain parallel during the compaction process.

The results of permeability tests on gyratory compacted specimens indicated that consistent permeability data could be obtained and reproduced in specimens compacted by this method. In addition, specimens could be prepared for the full range of densities desired in the permeability study. Therefore, the gyratory compaction method was used in preparing specimens for the permeability tests.

Specimen Size, Water Content, and Saturation

A study by Lambe (15) indicated that to obtain valid results, permeability tests should be made using specimen diameters 15 to 20 times greater than the maximum

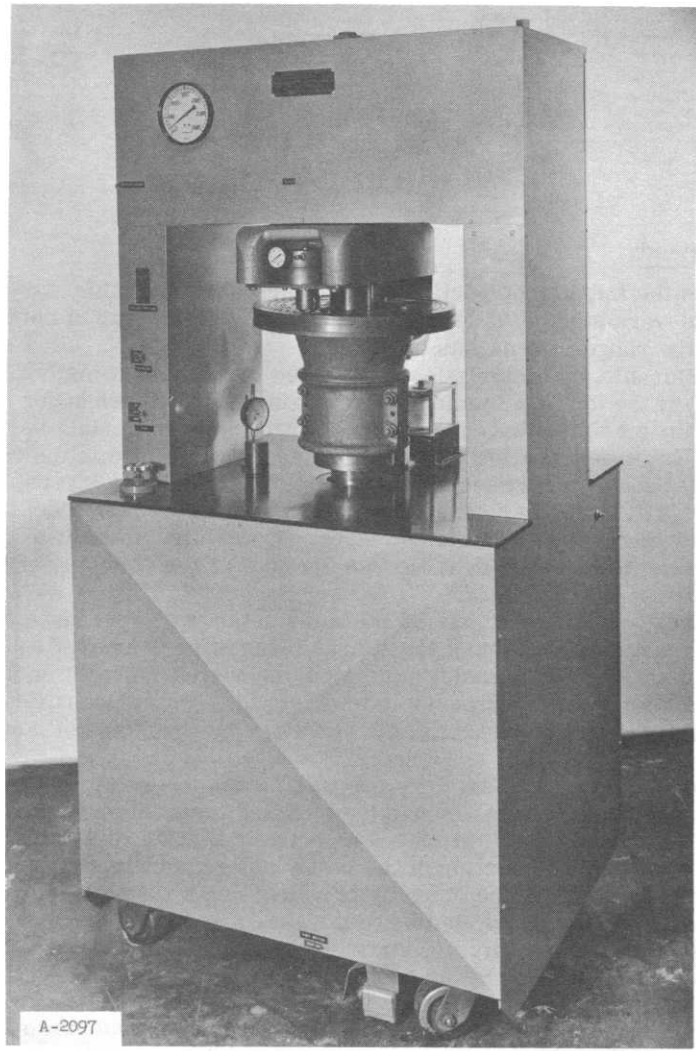


Figure 3. Gyratory compaction apparatus.

particle size. In the one 6-in. and one 12-in. diameter specimen of gradation 1 with a maximum particle size of 1 in., prepared by vibratory compaction, the specimen diameters were 6 and 12 times the maximum particle size respectively. Although the permeability for the 12-in. diameter specimen was 35 times that for the 6-in. diameter specimen, the comparison could not be considered definitive because of excessive segregation of particle sizes. It would also have been desirable for the specimen height to have been greater than 5 in. so that internal head losses could have been measured, using piezometers in the sides of the specimen mold. This would have eliminated the error, in computing the hydraulic gradient, from effects of entrance and exit head losses. Although it was not possible to obtain a specimen larger than 6 in. in diameter or higher than 5 in. in diameter with the gyratory compactor, this compaction method was used because it provided the best means of preparing uniform specimens.

Attempts to determine the effect of molding water content on permeability were inconclusive because of free drainage during impact and gyratory compaction and the dominant effect of segregation in vibratory compaction. However, in the process of

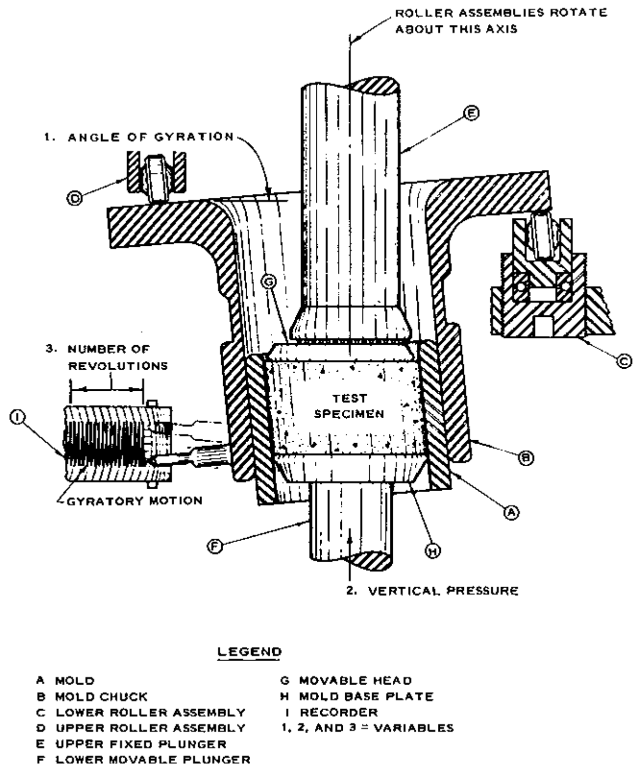


Figure 4. Gyratory mechanism.

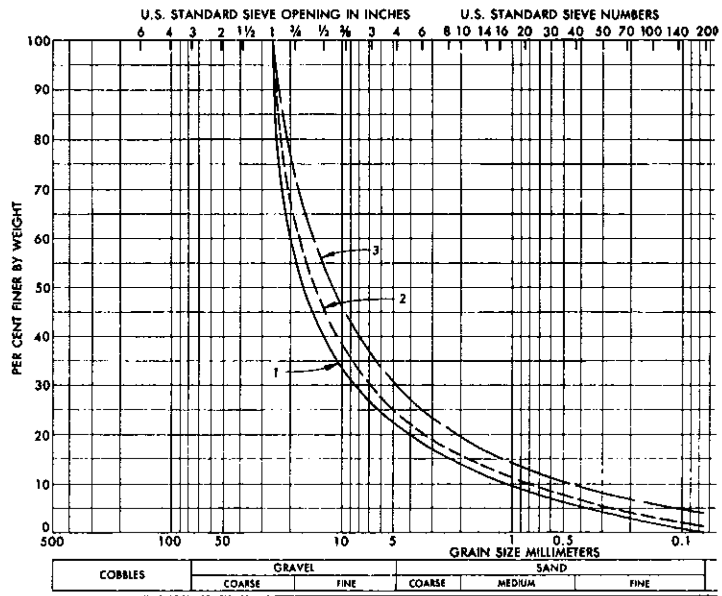
handling the material during preparation of specimens and during compaction, it was found that the tendency for particle segregation when placing material in the mold and drainage of water during gyratory compaction were minimized when specimens were prepared at or slightly below optimum water content. Therefore, all specimens prepared for permeability tests reported in this study were compacted at water contents near the optimum water content. The presence of entrapped air can have a significant effect on permeability results. To minimize this effect, all permeability specimens were submerged in de-aired water under an initial head of 8 in. until no head differential existed.

PERMEABILITY TESTS

Constant-head permeability tests were performed on specimens of gradation 1, 2, and 3 materials. Because of the low permeability of gradation 4 material, falling-head permeability tests were performed on this material. Effective porosity determinations were made after completion of the permeability test.

Preparation of Specimens

Each specimen of the four gradations of base-course material was prepared individually by blending predetermined amounts of material obtained from storage bins at WES. No material was reused after once being compacted. The amount of material blended at any one time was that weight needed to provide a single specimen, 6 in. in diameter and 5 in. high, at the desired density. Water was added to the dry material to provide a water content near optimum. The water content aided in preventing segregation of particle sizes during preparation of specimens. The moist material was placed in a moistureproof container and allowed to cure overnight. Prior to being



1. BEFORE COMPACTION
2. AFTER GYRATORY COMPACTION TO 137.5 PCF
3. AFTER IMPACT COMPACTION TO 137.8 PCF

Figure 5. Comparison of gradation curves for gradation 1 before and after gyratory and impact compaction.

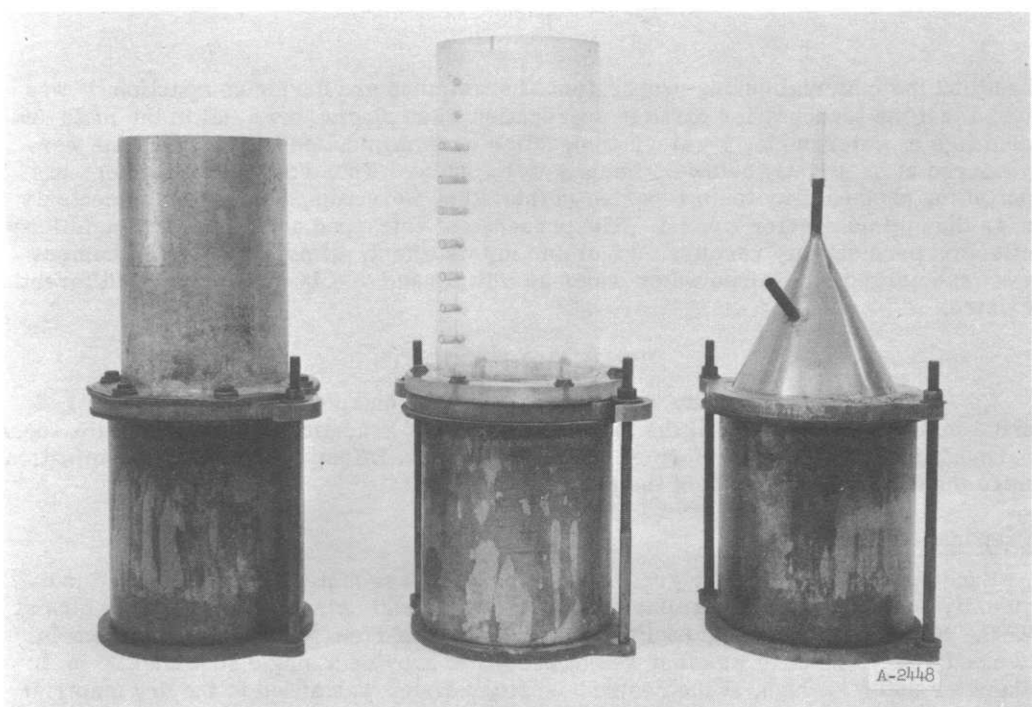


Figure 6. Molds containing permeability specimens with metal standpipe, plastic cylinder, and metal cone.

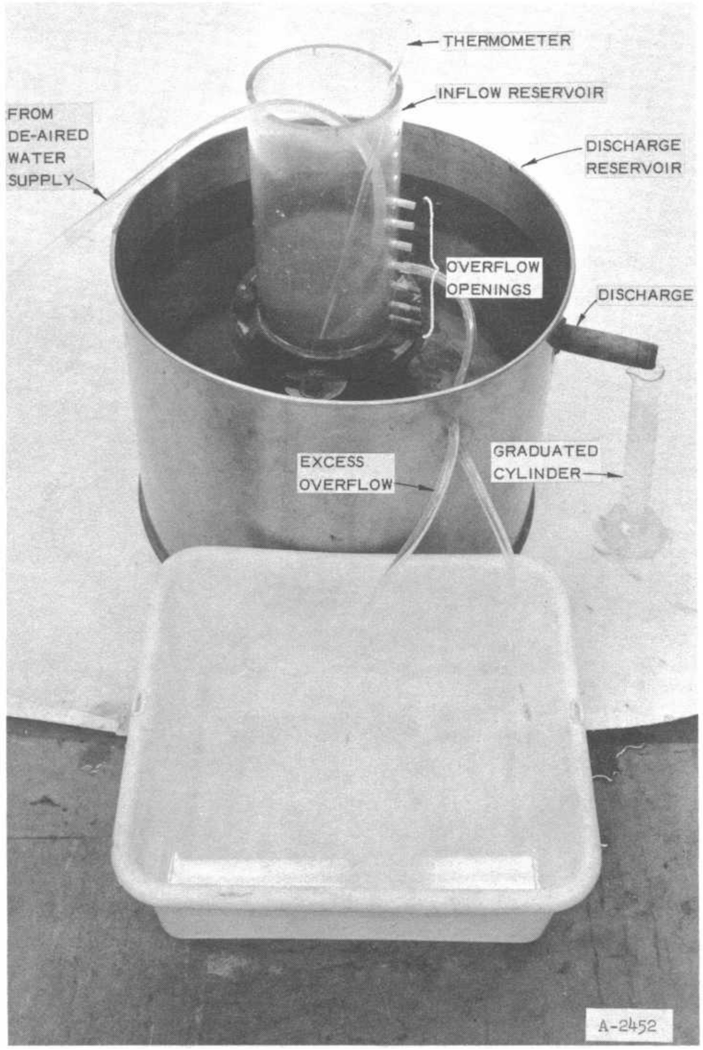


Figure 7. Constant-head permeability test apparatus.

spooned into the mold for compaction, the moist material was placed in a large pan and thoroughly remixed.

After compaction, the dry density of each specimen was computed based on the actual measured height of the specimen and the predetermined dry weight. No. 10 size screens were placed on both the top and bottom faces of each specimen. Crushed limestone, composed of particles between the ⅜ in. and No. 4 sieve sizes, was used as a filter in the space formerly occupied by the upper plate of the gyratory compactor, and it was retained there when the mold was inverted into the test position by a perforated base plate with $^{3}/_{32}$-in. diameter holes spaced at ¾-in. centers.

The mold containing the specimen was then inverted and a watertight sheetmetal standpipe was attached (left, Fig. 6). The apparatus was then placed in a tank containing de-aired water. The water in the tank was about 3 in. higher than the top of the specimen, which permitted the 5-in. high specimen to saturate from the bottom up under an initial head of 8 in. Each specimen remained in the reservoir until the water levels of the standpipe and reservoir equalized; a minimum saturation period of 16 hr was generally used. Several of the denser specimens of gradations 3 and 4 required up to 7 days to saturate.

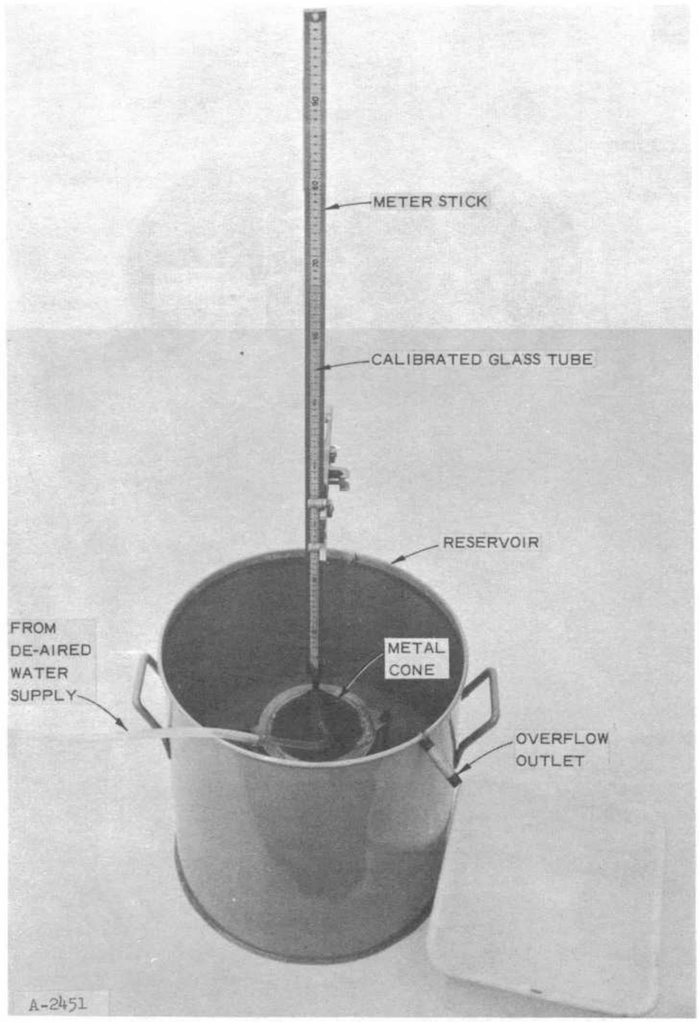


Figure 8. Falling-head permeability test apparatus.

Constant-Head Permeability Test

The metal standpipe used during the saturation process was replaced by a clear Lucite cylinder (center, Fig. 6) that served as the constant-head reservoir. Overflow openings spaced vertically at 1-in. centers in the inflow reservoir permitted testing at various heads. For falling-head permeability tests, a metal cone was used (right, Fig. 6).

The constant-head permeability test apparatus set up for testing is shown in Figure 7. De-aired water was supplied to the inflow reservoir at a rate necessary to maintain the desired constant head. The flow through the specimen was in the downward direction for all tests. The test head was the vertical distance between the water levels of the two reservoirs, and the hydraulic gradient was computed by dividing the test head by the height of specimen. A thermometer was placed in the inflow reservoir to indicate the water temperature throughout the tests.

The discharge was usually determined volumetrically by measuring the overflow from the discharge reservoir in a graduated cylinder. However, in several tests, measurements of flow were determined by weight rather than volume because of the large quantity of flow.

Each permeability test was continued until the flow became relatively constant or for 15 min, whichever was longer. The discharge was generally recorded for each 1-min interval. The discharge recorded during the final 5 min under constant flow conditions was used to calculate the coefficient of permeability (12) of the specimen.

Each specimen was subjected to a constant-head flow under successively increasing hydraulic gradients. The hydraulic gradient in the initial test was approximately 0.1. The head was then increased by 1- or 2-in. increments for each subsequent test to a maximum hydraulic gradient of approximately 1.0. Each specimen was tested under a minimum of five different hydraulic gradients. A few tests were performed in which the hydraulic gradient was first increased in increments as described and then reduced to lower values.

Falling-Head Permeability Tests

Falling-head permeability tests were performed on specimens of gradation 4, since no measurable discharge of permeating water was obtained during preliminary constant-head tests performed on this material. The metal standpipe used during saturation of specimens was replaced by an inverted metal cone. The setup for the falling-head permeability tests is shown in Figure 8. A glass tube of known constant, cross-sectional area was attached by a flexible tube to the opening at the apex of the cone just above the water level of the reservoir. The glass tube was mounted on the face of a meter stick, a portion of which was submerged in the reservoir. De-aired water was introduced into the system through the opening on the side of the cone until it flowed freely from the open end of the glass tube, exhausting all air from the cone and tube. An overflow outlet was provided to maintain the reservoir at a constant elevation.

To begin the test, the de-aired water supply was shut off at the opening on the side of the inverted cone, and the time required for the water level in the tube to drop from the original head, h_o, to a final head, h_f (interval h_o to h_f), was recorded. The time required for the water level to drop from the original head, h_o, to an intermediate head, h_i (interval h_o to h_i), was also recorded. The intermediate head h_i was chosen equal to $\sqrt{h_o h_f}$. Each specimen was tested at three head differentials (intervals h_o to h_f) of 40, 20, and 10 cm. Tests at each head differential were performed a minimum of five times, and the elapsed times were averaged for final computations (12). The water temperature was recorded immediately before each test.

EFFECTIVE POROSITY DETERMINATIONS

Upon completion of the permeability tests on each specimen, data for determining the effective porosity were obtained by removing the specimen from the reservoir and allowing it to drain freely for approximately 24 hr. After this time, a representative water content was taken of the drained specimen.

The permeability of a specimen denotes its ability to conduct water, but gives no indication of the total volume of water that can be drained from the material. Not all water contained in a given specimen can be removed by gravity flow, since water retained as thin films adhering to the soil particles and held in the voids by capillarity will not drain. Consequently, to determine the volume of water that can be removed from a soil in a given time, the effective porosity must be known in addition to the permeability. The effective porosity, n_e, is defined as the ratio of the volume of the voids that can be drained under gravity flow to the total volume of the soil mass and can be expressed as:

$$n_e = 1 - \frac{\gamma_d}{G_s \gamma_w} (1 + G_s w_e) \tag{1}$$

where

γ_d = dry density of the specimen,
G_s = specific gravity of solids,
γ_w = unit weight of water, and
w_e = effective water content expressed as a decimal fraction relative to dry weight.

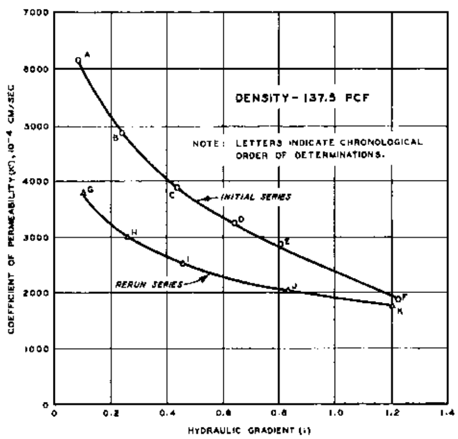


Figure 9. Effect of test sequence on coefficient of permeability (uncorrected for temperature), gradation 1, specimen 8.

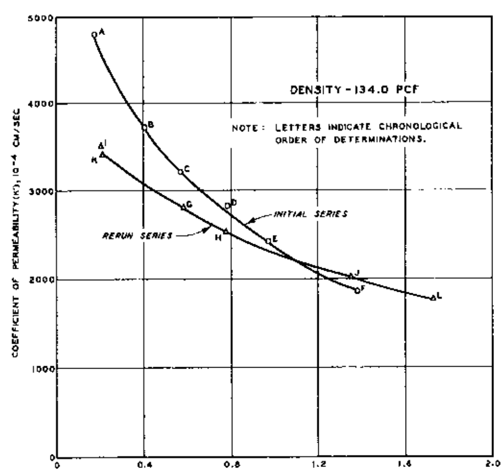


Figure 10. Effect of test sequence on coefficient of permeability (uncorrected for temperature), gradation 1, specimen 5.

The procedure for determining the effective porosity of a specimen was arbitrary since there is no accepted standard laboratory test procedure. Visual observations indicated that a 24-hr drainage period was sufficient to allow all water that was possible to drain by gravity from the permeability test specimen.

EFFECTS OF PARTICLE REARRANGEMENT AND TEMPERATURE

In preliminary constant-head permeability tests, it was found that internal rearrangement of particles contributed to the change in permeability with hydraulic gradient, but that variation in temperature of permeating water apparently had little effect. Studies were conducted to determine the influence of these factors.

Particle Rearrangement

Internal rearrangement of particles under seepage pressures may cause clogging of some seepage paths, and densification of the entire specimen may occur. Under condition of laminar flow, such arrangement would probably be insignificant, but under turbulent flow conditions, considerable rearrangement could occur with significant effect on permeability. This phenomenon was indicated in tests on gradation 1 when permeability determinations were first made at successively higher hydraulic gradients, then the hydraulic gradient was reduced to a low value and permeability determinations were then made under a second set of increasing gradients. Figure 9 shows the variation of the permeability values of the rerun series from the initial series of determinations for a specimen with dry density of 137.5 lb/cu ft. This plot appears to demonstrate that some internal rearrangement of particles involving clogging of seepage channels took place progressively up to the highest hydraulic gradient of the initial series, causing rerun permeability values to be lower than corresponding values of the initial series up to the highest hydraulic gradient, at which point the initial and rerun determinations coincide. No rearrangement of particles was indicated by similar tests on the densest of the specimens tested. For the less dense specimens, it was found that the permeability values obtained in the rerun series could be duplicated in subsequent determinations as long as the specimen was not subjected to hydraulic gradients greater than the previous maximum gradient. This is illustrated by results of initial and rerun series of tests shown in Figure 10. Note the chronological order of tests. The variations of permeability with hydraulic gradient in the repetitive determinations on the less dense specimens indicated that flow conditions were in the turbulent range, while laminar flow conditions were indicated in the dense specimens, since no change in permeability occurred with increase in hydraulic gradient.

The possibility was investigated that densification of the specimen may have occurred from seepage forces that developed as a result of water permeating through the specimen. The height of specimen before and after permeability testing was carefully measured on three specimens of gradition 1 at low, medium, and high densities to determine if any changes in volume had occurred during the tests. Results indicated that there was no increase in density of the specimen during the tests.

To determine if any significant migration of fines from the top to the bottom of the specimen was occurring during the initial series of tests, grain-size analyses were performed after completion of tests on the upper and lower halves of three specimens of gradation 1 material and five specimens of gradation 2 material. Grain-size distribution curves of the upper and lower halves of specimens of gradations 1 and 2 after permeability testing showed no noteworthy migration of fines to the lower half of the specimen. The gradations of the two halves also compared favorably with the original gradation of the specimen. Grain-size distribution curves obtained before and after completion of permeability tests for the filter material used between the base of the specimen and the perforated plate showed no significant migration of fines from the specimen into this material during the tests.

Temperature of Permeating Water

It is standard practice to report the coefficient of permeability corrected to a base temperature of 20 C by using the standard correction factors for viscosity of water. However, these correction factors are valid only for conditions of laminar flow. As test results in this study indicated that flow was not laminar in other than the denser specimens of gradations 1, 2, and 3, results were not corrected for temperature.

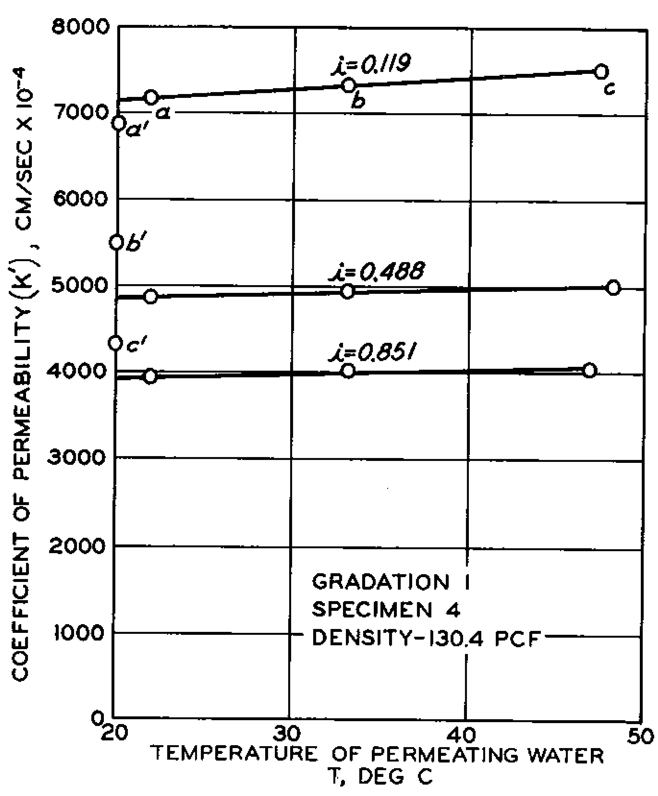


Figure 11. Effect of temperature on coefficient of permeability, gradation 1, specimen 4.

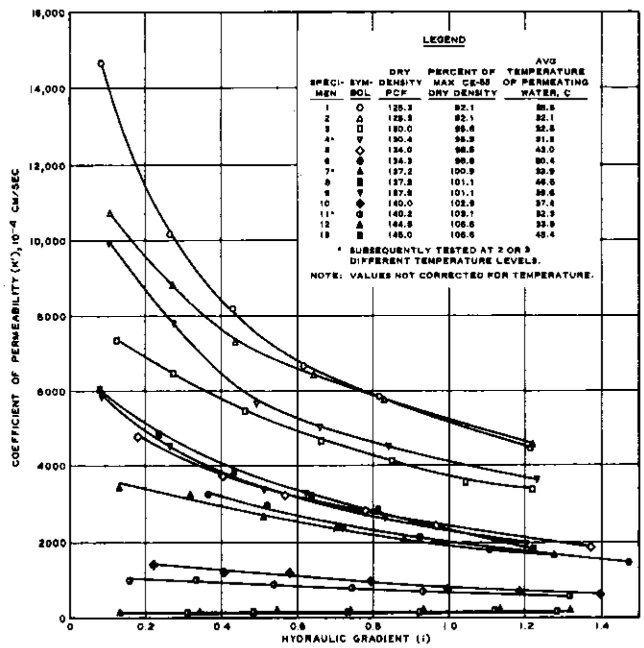


Figure 12. Coefficient of permeability vs hydraulic gradient for different densities, gradation 1.

To investigate the effects of temperature on permeability values, specimens of gradations 1, 2, and 3 compacted to low, medium, and high densities were tested at three different heads with permeating water at several different temperatures ranging from 19 to 48 deg. These determinations followed the initial permeability determinations in order to minimize the effect of rearrangement of particles during the tests.

Specimen 4, gradation 1, was compacted to 130.4 lb/cu ft, a low density for this material. The average temperature of the permeating water for the initial test series was 31.5 deg; the average temperatures for the subsequent series were 22, 33, and 47 deg. The measured permeabilities and the values of one set of tests corrected for the viscosity of the water at test temperature are shown in Figure 11. The difference between the points on the upper solid-line curve marked a, b, and c and the points marked a′, b′, and c′ is due to the use of temperature correction factors which cause divergence of data, and the fact that the uncorrected permeability data indicate no appreciable effect of temperature variations.

PERMEABILITY TEST RESULTS

Constant-Head Tests

A plot of the uncorrected permeability coefficient, k′, vs hydraulic gradient, i, is shown in Figure 12 for constant-head tests on gradation 1 material at different densities. Note that k′ is used to distinguish this coefficient from Darcy's coefficient of permeability, k, because laminar flow conditions did not generally apply. As a result, values of k′ are uncorrected for viscosity of the permeating water. Variation in k′ with i occurs in the majority of tests, and the magnitude of this variation decreases with increase in density as laminar flow conditions are approached. Plots of k′ vs i for gradations 2 and 3 are shown in Figures 13 and 14. The relationship of k′ and γ_d for gradation 1 is shown in Figure 15 in which curves relating k′ and dry density γ_d for selected values of i from Figures 12, 13, and 14 are plotted. Plotted values represent averages for specimens of approximately equal density. Figure 16 shows a similar relationship for gradation 2 and Figure 17 for gradation 3. These plots of k′ vs γ_d indicated that laminar flow was achieved or at least approached only in specimens

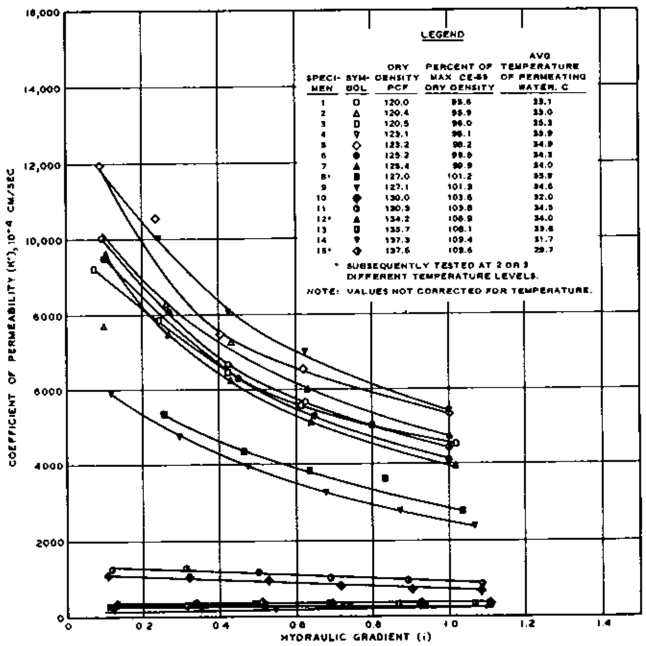


Figure 13. Coefficient of permeability vs hydraulic gradient for different densities, gradation 2.

of gradations 1 and 2 compacted to densities greater than approximately 105 percent of their respective CE-55 maximum dry density, and in specimens of gradation 3 compacted to densities greater than approximately 100 percent of CE-55 maximum dry density.

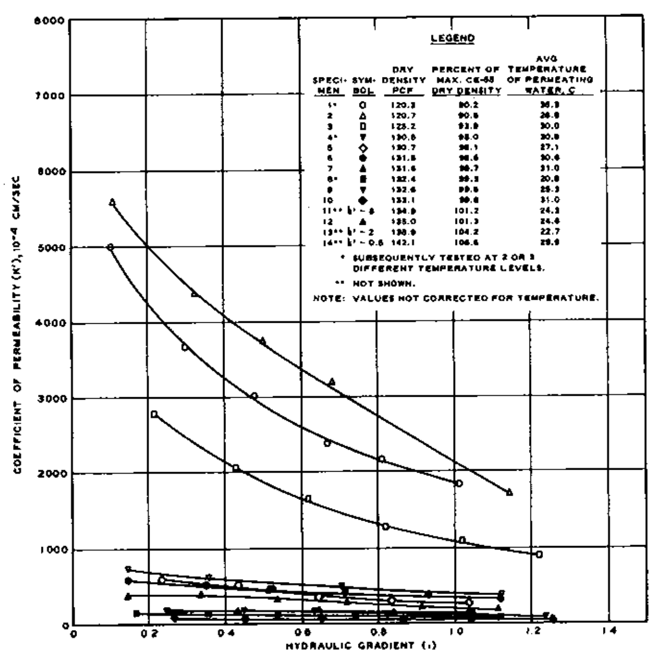


Figure 14. Coefficient of permeability vs hydraulic gradient for different densities, gradation 3.

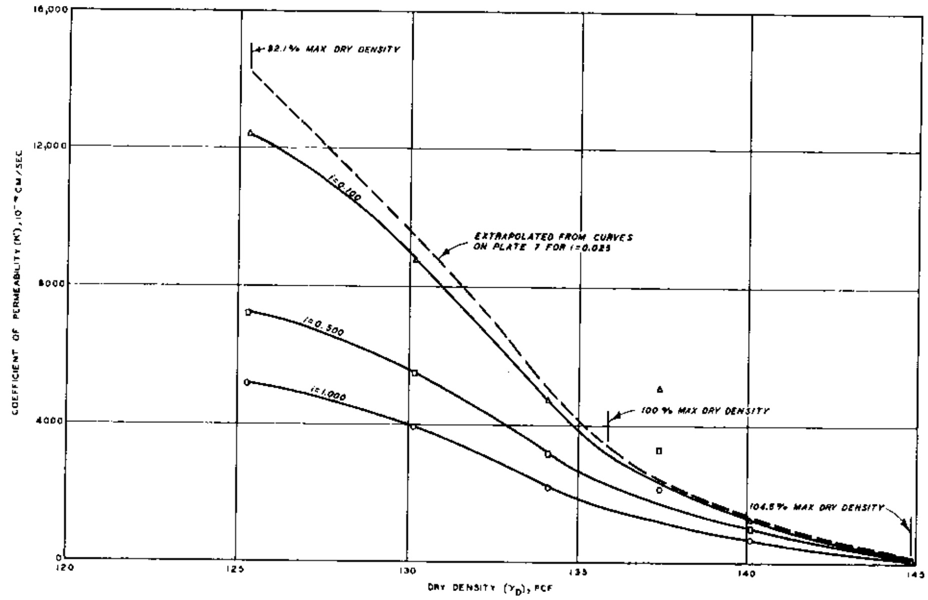


Figure 15. Coefficient of permeability vs dry density, gradation 1.

Falling-Head Tests

The results of falling-head permeability tests are not considered to be as reliable as results of constant-head tests and should not be used when the constant-head test can be used. Because of the small quantity of water permeating the specimens and the ratio of the cross-sectional areas of the specimen to the standpipe, small leaks, errors in measurements, movement of fines, incomplete saturation, or permeating

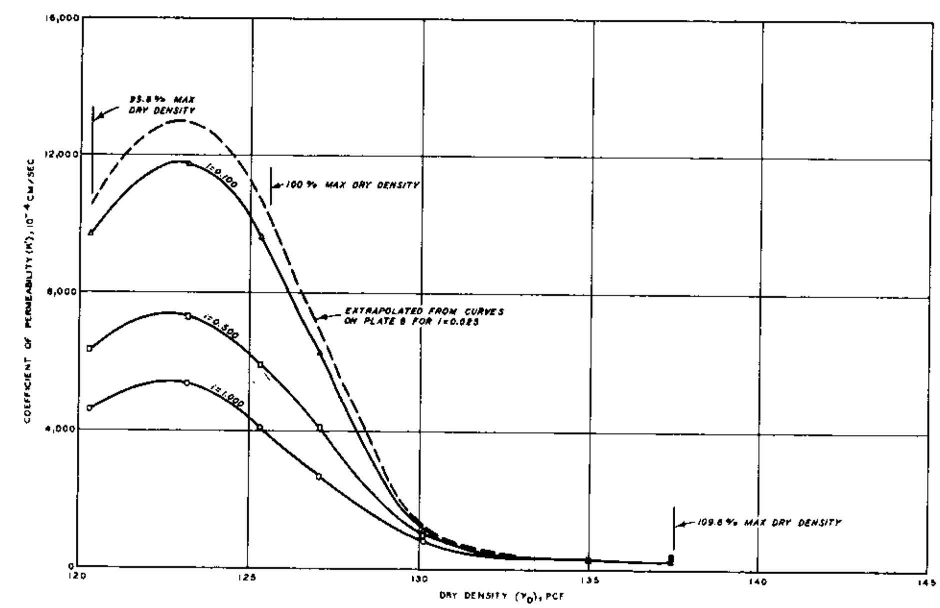


Figure 16. Coefficient of permeability vs dry density, gradation 2.

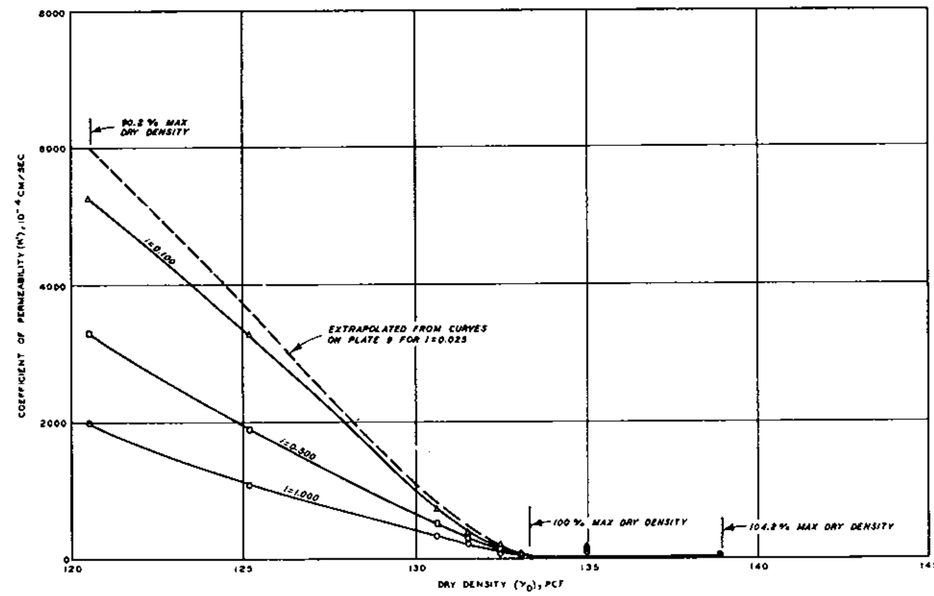


Figure 17. Coefficient of permeability vs dry density, gradation 3.

water not sufficiently de-aired can have much more significant effects on results than in constant-head tests. To reduce the chance of errors from these sources, tests were repeatedly run using the same head differential, and tests were also performed using different head differentials, as previously mentioned.

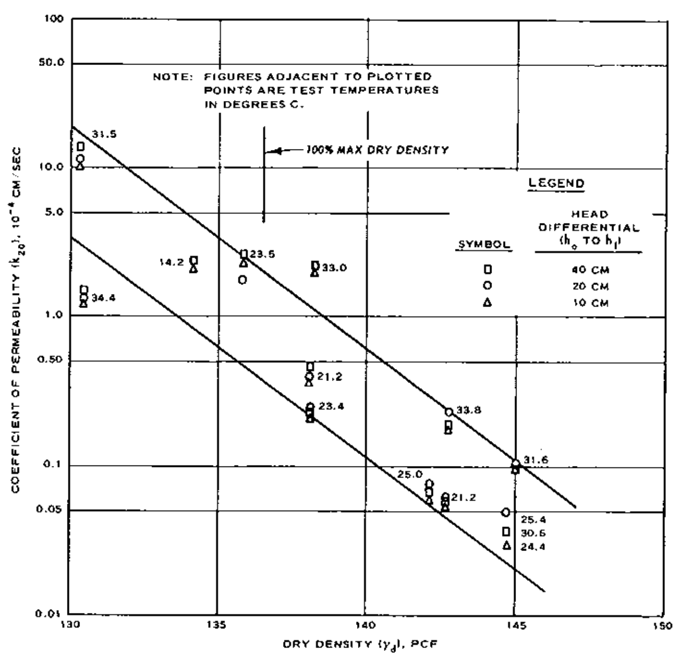


Figure 18. Coefficient of permeability vs dry density, gradation 4.

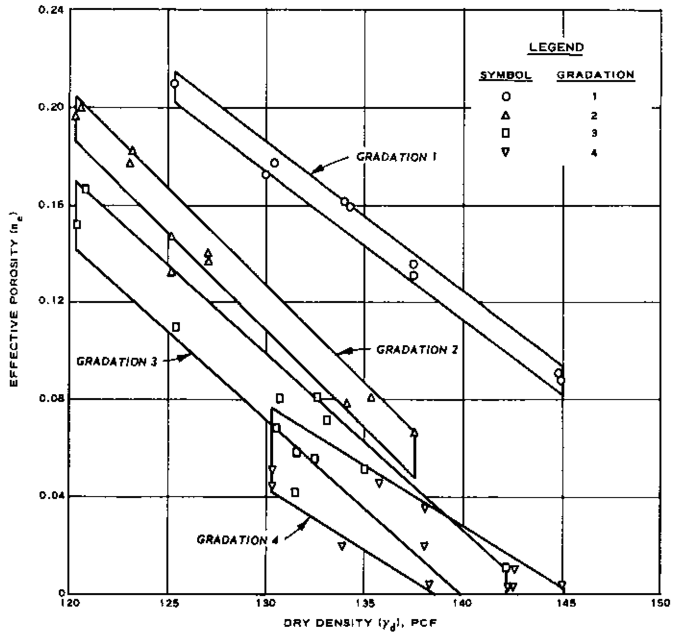


Figure 19. Effective porosity vs dry density.

Upon completion of the permeability tests on each specimen, values of coefficient of permeability were corrected for temperature by the standard viscosity ratio correction factor, R_T. The variation of permeability with dry density is shown in Figure 18. A band was drawn to bracket the plotted points; a variation in permeability of a factor of about 5 (½ log cycle) is indicated for a given density. This variation of test results is considered reasonable for falling-head permeability tests performed on individual specimens of this type of material. The three permeability values plotted at each density show the variation in results that occurred in performing the tests at various head differentials (interval h_o to h_f). This variation is relatively small when compared to variations obtained for different specimens compacted to the same densities.

EFFECTIVE POROSITY TEST RESULTS

The results of the effective porosity determinations made for the four materials tested are plotted as effective porosity vs dry density in Figure 19. The results indicate a range of effective porosity values for a particular density, and a band is drawn bracketing the data obtained for each gradation tested.

The data indicate that effective porosity decreases with increase in density and with increase in the percentage of small particle sizes. This is logical since in the finer gradations, a greater particle surface area is provided for the water to adhere to and in the denser specimens, the size of the voids are decreased and greater capillary forces develop. Consequently, less water drains from the material.

The test data can be plotted in the more familiar terms of degree of saturation after drainage, S_e, vs dry density. This is shown in Figure 20 and indicates the relation between dry density and the degree of saturation of the specimens after drainage. These plots show that the degree of saturation of the denser specimens of gradations 3 and 4 materials after drainage approaches 100 percent. Thus, only a very small amount of the total water in these materials will drain by gravity when the materials are highly compacted. The degree of saturation after drainage is inversely related to effective porosity. Consequently, S_e decreases with decrease in density and with decrease in the percentage of small particle sizes.

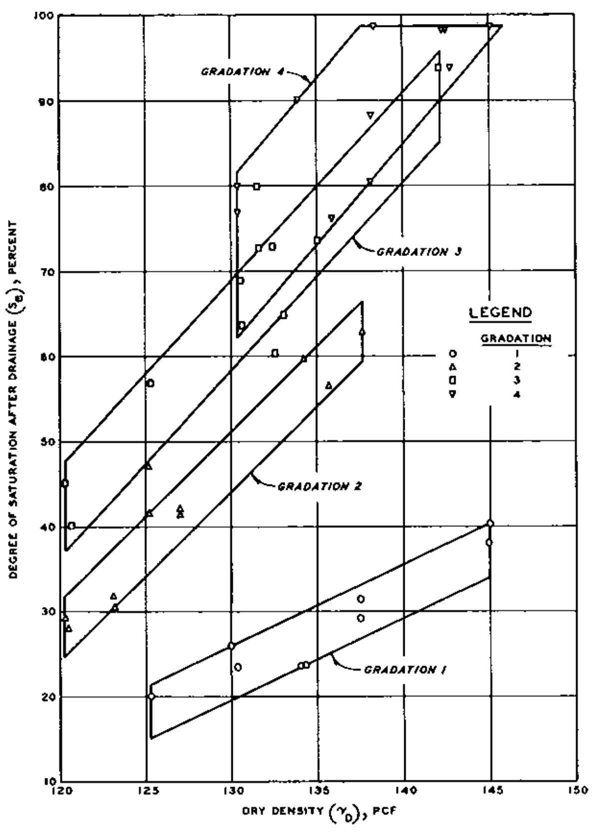


Figure 20. Degree of saturation after drainage vs dry density.

APPLICATION OF RESULTS

As previously mentioned, U.S. Corps of Engineers criteria for design of base-course subdrain systems for airfields require that a degree of drainage of 50 percent be obtained in not more than 10 days. To determine if the drainage criteria can be met under field conditions, the dimensions of the base course to be drained must be considered in addition to the drainage characteristics of the base-course material (Fig. 21). The equation used to determine the time required for a saturated base course to reach a degree of drainage of 50 percent when the necessary dimensions and characteristics are known is

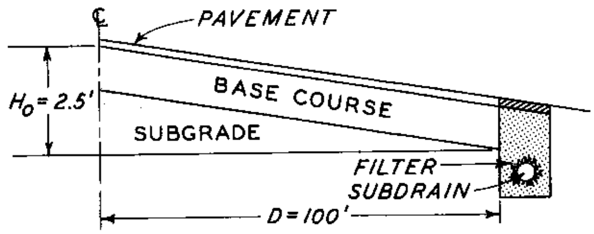
$$t = \frac{n_e D^2}{2(1440)\, k\, H_o} \tag{2}$$

where

- t = time for 50 percent drainage, days;
- n_e = effective porosity;
- D = horizontal drainage distance, ft;
- H_o = dimension as shown in Figure 21; and
- k = coefficient of permeability, ft/min (2 x cm/sec = ft/min).

However, the following assumptions must be made when applying this equation:

1. The base is initially 100 percent saturated.
2. No recharge occurs once drainage begins.



TIME FOR 50 PERCENT DRAINAGE:

$$t = \frac{n_e D^2}{2(1440) k H_0}, \text{ DAYS}$$

n_e = EFFECTIVE POROSITY

D = HORIZONTAL DRAINAGE DISTANCE, FT

H_0 = TOTAL HYDROSTATIC HEAD, FT

k = COEFFICIENT OF PERMEABILITY, FT/MIN

GRADATION	MAX CE-55 DRY DEN. LB/CU FT	k FT/MIN $\times 10^{-4}$	n_e	t DAYS	S_e %
1	136	6,400	0.13	0.3	28
2	126	20,000	0.15	0.1	40
3	133	100	0.06	8.3	70
4	136	2	0.03	208.0	85

Figure 21. Effect of gradation and dry density on time for 50 percent drainage.

3. The subgrade is impervious and drainage occurs mainly in the transverse direction of the base-course material.
4. The coefficient of permeability and effective porosity of the base-course material are constant and the same in all directions for the existing conditions.

The theoretical hydraulic gradients under which drainage occurs will change from $H_0/0$ (or infinity) at the instant drainage begins to H_0/D at the instant a degree of drainage of 50 percent is reached. The effective porosity, as determined by the method previously discussed, is independent of the hydraulic gradient, but the permeability of base-course materials was found in the tests reported herein to depend on the hydraulic gradient for densities less than about 98 to 103 percent of CE-55 maximum dry density. Consequently, assumption 4 is not completely satisfied because of the variable permeability. However, the assumption can possibly be satisfied conservatively for design purposes by determining the permeability of the material at an effective hydraulic gradient, i_e, of $2H_0/D$. The effective hydraulic gradient will indicate a permeability value lower than that indicated by either the slope of the base course, s, or the final theoretical hydraulic gradient, H_0/D. The effective gradient was established by dividing the total hydrostatic head, H_0, by the average horizontal drainage distance, D/2. Assumptions 1, 2, and 3 are accepted as valid, although they are idealized conditions.

In an effort to determine if the drainage criteria could be met for a runway having subdrain systems located only along the edges of the pavement, computations were made using the equation for t and the results from the permeability and effective porosity tests on the four materials tested. The results are shown in Figure 21. Gradations 1, 2, and 3 meet the criteria at 100 percent of CE-55 maximum density, but gradation 4 does not. At higher densities, gradation 3 would not meet the criteria and the degree of saturation would also increase. Gradations 1 and 2 would meet the criteria at much higher densities.

CONCLUSIONS

The results of the study show that the drainage characteristics of coarse-grained materials can be determined with sufficient reproducibility by following the test procedures described herein to provide a means of evaluating the effect of density and gradation on permeability in developing design data. It is recognized that the ratio of the mold diameter to the maximum soil particle size and specimen height may have been too small in these tests and that the measured k′ values were undoubtedly affected to some degree. However, the advantages of the gyratory compaction method in providing uniform specimens are considered to outweigh the disadvantages of the mold-size limitation imposed by its use.

To obtain reliable laboratory test results that will reflect the actual field conditions, the permeability test specimens of base-course material should be prepared and compacted to simulate the field conditions as closely as possible.

Laminar flow cannot be anticipated in permeability tests on many base-course materials; thus, expected field conditions for hydraulic gradient and temperature of permeating water should be approximated in the laboratory test. Where field conditions are not known, a series of tests should be performed to bracket estimated field conditions. Where a series of tests is to be performed, testing should begin at the lowest hydraulic gradient to minimize effects of particle rearrangement on test results.

Computations using the results of this study, present drainage design criteria in TM 5-820-2, and the dimensions of a typical airfield runway indicate that it is theoretically possible for a base-course material to meet the drainage criteria in TM 5-820-2 and still remain at a high degree of saturation when the dry density is equal to or greater than 100 percent of CE-55 maximum dry density. This is possible when the effective porosity of a material approaches zero. The effective porosity can approach zero in a highly compacted material containing as low as 5 percent of fines. A continued high degree of saturation of a material compacted to 105 to 110 percent of CE-55 maximum dry density may not materially affect the strength of the material; however, excess pore water pressure may develop under traffic and cause pavement distress. The presence of water in the material may also increase the possibility of damage to an overlying pavement from frost penetration.

REFERENCES

1. Casagrande, A., and Shannon, W. L. Base Course Drainage for Airport Pavements. Trans. ASCE, Vol. 117, pp. 792-820, 1952.
2. Subsurface Drainage Investigation 1945-1956, Comprehensive Report. U. S. Army Engineer District, Boston, Arctic Construction and Frost Effects Laboratory TR-5, Sept. 1946.
3. Terzaghi, K. Effect of Minor Geological Details on the Safety of Dams. Amer. Inst. of Mining and Metallurgical Eng.
4. Investigation of Filter Requirements for Underdrains. U. S. Army Engineer Waterways Experiment Station, Tech. Memo. No. 183-1, Vicksburg, Miss., Nov. 1941.
5. Field and Laboratory Investigation of Design Criteria for Drainage Wells. U. S. Army Engineer Waterways Experiment Station, Tech. Memo. No. 195-1, Vicksburg, Miss., Oct. 1942.
6. Sherman, W. C. Filter Experiments and Design Criteria. U. S. Army Engineer Waterways Experiment Station, Tech. Memo. No. 3-360, Vicksburg, Miss., April 1953.
7. Drainage and Erosion Control: Subsurface Drainage Facilities for Airfields. U. S. Army, Office, Chief of Engineers, TM 5-820-2 (EM 1110-345-282), Aug. 1965.
8. Nettles, E. H., and Calhoun, C. C., Jr. Drainage Characteristics of Base Course Materials, Laboratory Investigation. U. S. Army Engineer Waterways Experiment Station, Tech. Rept. No. 3-786, Vicksburg, Miss., July 1967.
9. Specifications for Military Construction, Graded-Crushed-Aggregate Base Course. U. S. Army, Office, Chief of Engineers, CE-807.07, Feb. 15, 1966.
10. Guide Specification for Military Construction, Stabilized-Aggregate Base Course. U. S. Army, Office, Chief of Engineers, CE-807.03, Feb. 15, 1966.
11. Test Method for Pavement Subgrade, Subbase, and Base Course Materials. U. S. Army, Office, Chief of Engineers, Military Standard 621A, Dec. 1964, changed by Notice 1, June 2, 1966.

12. Laboratory Soils Testing. U. S. Army, Office, Chief of Engineers, EM 1110-2-1906, May 10, 1965.
13. McRae, J. L. Development of the Gyratory Testing Machine and Procedures for Testing Bituminous Paving Mixtures. U. S. Army Engineer Waterways Experiment Station, Tech. Rept. No. 3-595, Vicksburg, Miss., Feb. 1962.
14. Feasibility Study of the Gyratory Machine for Testing Soils. U. S. Army Engineer Waterways Experiment Station, Miscellaneous Paper No. 3-474, Vicksburg, Miss., Feb. 1962.
15. Lambe, T. William. Soil Testing for Engineers. John Wiley and Sons, Inc., New York, 1951.

Experiments With Filter Materials for Subdrains

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This report deals with work with agricultural tile drainage systems; their requirements differ in many respects from the requirements for highway subdrains. There are also environmental differences: agricultural tile drains are required predominantly in basin lands, and in valley (flood plain) lands where there is considerable stratification. Tile drains are required rather infrequently in uplands and terrace lands. Major highways now largely avoid basin lands, and their shift more to the terraces and uplands would be beneficial to man's environment. Some subdrains for highways might be required on the terraces, but the main need should be with unstable upland slopes.

● THIS work was undertaken from the viewpoint of improving the effectiveness and efficiency of agricultural tile drainage systems. The environment under which such drains are placed, and the criteria for placement design, are normally quite different than subdrainage as required for highways. Nevertheless, the findings of one discipline can well prove useful to another discipline, and it is hoped that some cross-fertilization might develop in this instance.

AGRICULTURAL SUBDRAINS

Purpose

Agricultural tile drains in all climates serve the function of: (a) improving the bearing power of the soil surface to permit tillage, planting, harvesting and other essential traffic with a minimum of compaction; and (b) providing aeration of the plant rootzone. Plants, for optimum growth, require a soil of low bulk density and, for most plants, a well-aerated rootzone from somewhere between 18 in. to at least 5 ft in depth. In arid and semiarid regions, where irrigation is regularly required, there is a third and often dominant function–to provide a rootzone low in salts. Soil waters and irrigation waters always contain some salts, and the plant rootzone is the concentration zone of those salts. Evapotranspiration (evaporation from soil and water surfaces, and transpiration from the leaves of plants) removes pure water and leaves salts behind. Maintenance of a low-salt rootzone implies the periodic application of sufficient water to accomplish some leaching of salts downward in excess of the water required to replenish the rootzone, and a water table (normally fluctuating) whose mean depth is sufficient to minimize the total upward capillary movement of moisture. The required mean depth of the water table usually ranges from about 4 to 6 it in and regions, depending on soil, crop, and tile spacing. The depth of the tile lines themselves is usually 1 to 2 ft greater than the mean water table depth.

Environment

The type of land where agricultural subdrains are required can be explained in accordance with the characteristics imparted to that land by its position on the landscape,

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as usually found in the southwestern United States. The upland soils are the primary or residual soils formed by the weathering of the underlying rock materials. They have varying amounts of profile development (a subsoil differing to some degree in texture and structure from the topsoil), depending on climate and relative rates of erosion. Most erosion occurs during major floods, and the sediments are carried downstream to the floodplain where deposition takes place. As flows emerge from the canyons, velocity decreases with a drop in gradient and floodwaters spread out over the floodplain. Although there is much textural mixing, increasing with an increase in density of the sediments in the suspension, the soils deposited in the valleys are normally of loamy medium texture, and of fine texture in the basins (largely clays, clays being defined as soils where the effective diameter of the particles is less than 2 microns). Slopes are so flat in the basins that drainage is impeded, and there is at least temporary ponding of the floodwaters, permitting finer sediments to settle out.

The floodplain soils–valley and basin–are called alluviums. They are usually called recent alluviums because deposition during floods has been rapid enough to prevent any "maturing" or development of a profile. Their relative uniformity to a considerable depth makes them invaluable as agricultural lands. These lands have good groundwater aquifers, and pumping the groundwater for a water supply normally keeps the water table well below levels that would call for subdrains. However, as the distance towards the basin lands increases, and as aridity, which is related to more stratification, increases, groundwater pumping ceases to be an effective water level lowering mechanism. It is in the basin and basin rimlands where subdrains are required.

Terrace lands are floodplain alluvium that, through geologic change, are no longer subject to deposition and are on a flat enough slope to avoid appreciable erosion. They have therefore matured to a variable extent–often having developed marked differences between topsoil and subsoil. In the southwestern climates, terrace subsoils commonly have much lower hydraulic conductivity than topsoils. After prolonged rain, water infiltrating the topsoil has wet up that topsoil and begins to develop a temporary perched water table which moves off laterally. These, and some upland soils with similar profile development, are far from being prime agricultural lands, but, because of night air drainage, they may be prized for certain agricultural specialty crops requiring relative freedom from frost. Subdrains at a depth to intercept the lateral flows may sometimes be required in the interests of soil aeration.

There are some agricultural subdrains required in upland and terrace lands. However, the dominant need is in basin lands, and in valley lands when there is considerable stratification. This is flat land and subdrains are usually on a grade of about 0.001, with pumps lifting the effluent several feet from sumps if a low enough outlet cannot be obtained. In many respects, there is lack of uniformity in such lands, but there is a great deal of uniformity in the type of land requiring agricultural subdrains. Relatively large areas are usually involved, all in the same environment requiring much the same treatment.

HIGHWAY SUBDRAINS

Environment

There are distinct advantages in getting highways and population concentrations out of the valleys and basins and onto the terraces and uplands. This relates to atmospheric pollution (1), avoidance of the flood hazard, and preservation of prime agricultural lands. In addition, terrace lands and rolling uplands provide, aesthetically, a much better environment for urban development. Sacramento, for example, was originally planned as a natural transportation hub, and no visible effort has been made to alter this. There would be marked advantages if the state would start planning its new capitol buildings for the terrace lands to the east. Planning for new highways tends to follow traditional patterns, yet the effects on urban trends when highways take a new route are readily apparent.

There should be a concerted effort, with all major highways, to avoid basin lands and to get out of the valleys as much as possible. As a rule, major highways can avoid basin lands and those parts of valleys requiring agricultural subdrains. Secondary highways on such lands, with compacted subgrade and surface seal, will continue to require parallel ditches or subdrains to intercept lateral flows. On terraces and uplands with soils having profile development, it is presumed that topsoil will be removed, or mixed with the subsoil and compacted. Therefore, subsurface drainage will be attained with interceptor drains parallel to the upstream side of the highway. Since these will be relatively shallow, they will most often be ditches rather than covered drains.

The major needs for highway subdrains would seem to be in upland areas of unstable slopes; unstable because of geologic conditions. The greatest hazards of landslides would seem to be in sedimentary formations where downsloping beds of shale or clay might be involved. Water seeps above the strata interfaces can provide lubrication, particularly where hydrostatic pressure exists. Any cuts and fills often adversely affect stability, and interception of seeps can improve stability.

In summary, most highway subdrains should require individual investigation, and not be susceptible to the rather uniform treatment that marks the agricultural drains. Furthermore, there can be significant differences in design for the two types.

ENVELOPES AND FILTERS

Almost all agricultural "tile lines" are made either of vitrified clay or of concrete[1] with water entering at the unsealed joints. Perforated and slotted plastic tile and perforated bituminized fiber pipe have been used, but these are all still experimental. One company is considering experiments with a highly porous plastic pipe having adequate structural strength to resist crushing. Porous concrete drainage pipe has been manufactured, but the writer does not recommend its use.

Most of such tile drainage lines are installed with nothing surrounding them but soil. In the humid regions, it is sometimes customary to select topsoil for the initial backfill in immediate contact with the pipe because it will have higher hydraulic conductivity than subsoil. However, in more recent years, it has often been found advantageous, in promoting inflow into the tile lines of humid regions, to surround the pipe with some type of "blinding material" or envelope. Use of such material allows the streamlines of flow to be normal to the pipe instead of converging at the joints. Further, the large pores in the material permit the water to move up to the tile at low velocity so that soil material is not disturbed or carried in the water.

The term "blinding material" generally refers to some type of vegetative residue such as tule trash or a crop residue. Engineers in Holland report that they have been successful with heather trash, and that peat has also been used. The Oxnard Plain in southern California seems to be the first area to use pea gravel extensively as an envelope material. Farmers in that former swampland originally had trouble with the tile lines silting up, but their solution was to connect the upper end of each tile line with an irrigation hydrant. By opening a slide gate on the side of a hydrant, any tile line could be flushed out from time to time. This system has functioned quite satisfactorily.

Tile drainage developed later in the Imperial Valley of California. Irrigation was by open ditch, making it impractical to do anything but plug each tile line at its upper end. At the same time, there was the problem of lines being plugged with soil material. It was probably in this valley more than anywhere else that the concept of a filter developed Pits were located that seemed to provide a reasonable filter of sand material. Actually, it was used as is, i.e., a "pit run" material, because the cost of washing or sizing would be prohibitive. By and large, it did a reasonable job. However, texture

[1] Clay tile is made in accordance with ASTM Designation C4, latest revision, "Standard Specifications for Vitrified Clay Drain Tile" and concrete drain tile in accordance with ASTM Designation C412, latest revision, "Standard Specifications for Concrete Drain Tile." In the western United States, concrete drain tile is made in accordance with ASTM Designation C118, latest revision, "Tentative Specifications for Concrete Pipe for Irrigation or Drainage." In the arid to semiarid regions where lines are 6 to 8 ft in depth, heavy duty pipe, as provided in C4 and C118, is specified. Where sulfates can be a problem, type II or type V cement is specified for C118 pipe.

was quite variable and some lines were found to silt up over a period of years. The writer made textural analyses of the filters used on such lines and of the material found on the inside of the lines. This material was very fine sand or a mixture of very fine sand and coarse silt. (Effective diameter of 50 percent of the material "finer than" ranged from 0.066 to 0.12 mm.) It was suggested that material conforming to the grading requirements of standard concrete sand (2) would make a satisfactory filter.

The soils of the valleys of southern California are badly stratified, and thin lenses or strata of a cohesionless material in the textural range of very fine sand to very fine sand and coarse silt are commonly encountered. There are, of course, both finer and coarser strata, but it is assumed that finer material, which contains clay, has enough cohesion to resist erosion, or is carried on through the tile lines. It is assumed that joint openings, as occasioned by irregularities, are too narrow to admit coarser material. Therefore, concern is simply for a rather narrow range of material, and there is no need to adjust the grading of the filter to the texture of the soil in which the tile is placed.

There is some evidence in the Imperial Valley, and considerable evidence in the Coachella Valley, of a gradual degradation in the effectiveness of tile lines. A pit run material somewhat similar to that used in Imperial is used in Coachella. It is known to contain fines, including a considerable amount of micaceous material. Gradual sealing apparently occurs in the joints. Laboratory studies were undertaken using Coachella soil and Coachella filter sand. The following are conclusions from that study:

1. Over a period of several months, and with a constant head of water on the soil surface, rate of flow from the tile does deteriorate with time. Prime deterioration is in the first one or two weeks.
2. Washing of the filter sand before placement resulted in considerably increased flow, but deterioration with time continued.
3. Surging of the laboratory tile was quite effective in improving flow, but did not eliminate the deterioration with time. Surging was a mild application of the way wells are developed, but done so as not to displace the filter.

Surging was then tried on two tile lines in the Coachella Valley. There was marked and immediate improvement in performance, but in about a week the tile lines were back down to the poor level of performance that existed before surging. The marked deterioration in performance may be limited to the Coachella Valley. No evidence of it has been observed in the San Joaquin Valley where filters are also used. Incidently, there is no firm evidence as yet that filters are required in the San Joaquin Valley, although there is evidence that envelopes are beneficial.

Design Criteria for Sand Filters

For sand to function as a satisfactory filter, it appears desirable to filter out cohesionless material in the D_{50} range of 0.06 to 0.12 mm, based on findings in the Imperial Valley. Employing the well-known F/A ratio (3, 4, 5, 6)

$$F/A = \frac{D_{50} \text{ of filter}}{D_{50} \text{ of aquifer}}$$

and the given range of D_{50} for cohesionless soil material, provides a basis for beginning the filter design. Qazi (7) and des Bouvrie (8) suggest the use of the standard deviation as an index of the range in gradation of the filter. They state: "Dixon and Massey (9) show that in a large normally distributed sample, the area enclosed by the ordinates through points at a distance of 1.645 (where σ is the standard deviation of the sample) on each side of the mean σ contains 90 percent of the total variates in the sample. Therefore, the standard deviation (σ) can be found from an accumulation grain size distribution plot on logarithmic probability paper (on which a normal distribution plots as a straight line), by dividing the interval between the 95 percent and the 5 percent size by 2 x 1.645 = 3.29."

Qazi and des Bouvrie present a chart (Fig. 1) showing minimum suggested values of the F/A - σ relationship to provide a satisfactory filter. Further, des Bouvrie (8) suggests the following minimum filter thicknesses with successful F/A - σ combinations:

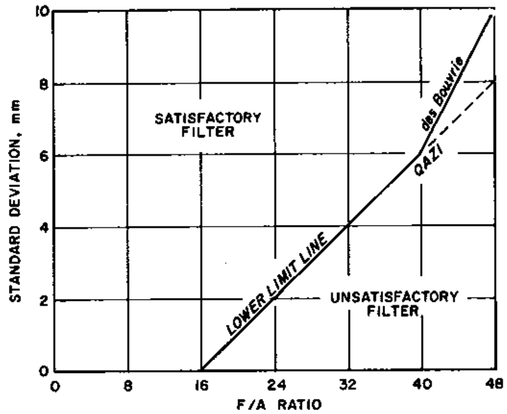

Figure 1. Lower limit line, as suggested by Qazi and modified by des Bouvrie, for a satisfactory filter.

1. 0. 5 to 1.0 in. for F/A ≤ 12;
2. 3 in. for F/A = 12 to 24;
3. 6 in. for F/A = 24 to 28; and
4. 9 in. for F/A = 28 to 40, and so on.

For reasonable economy, the minimum filter thickness should not exceed 3 in., although this may be more important with agricultural tile drains than with highway subdrains.

Referring again to standard concrete sand (2), by adhering to the customary grading restrictions and the given criteria, F/A can range from 4 to 20 and σ from 0.54 to 1.41 mm. Some concrete sand might therefore be unsatisfactory. Costs to satisfy the needs of highway subdrains would be less of a factor, and complete success can be assured with the additional restrictions that $D_{50} \leq 1.0$ mm and σ ≥ 1.0 mm, or a specially graded filter could be designed and specified in accordance with the restrictions. This might well be preferred with highway subdrains. In the writer's experience and laboratory trials, material adhering to these criteria have been quite satisfactory. All of the Imperial Valley material that failed as a filter did not adhere.

Glass Fiber Mat Filters

A commercial organization was installing perforated bituminized fiber pipe in Imperial Valley with a 1-in. thick by 12-in. wide fiber glass mat placed over the top of the pipe. Perforations were on both sides of the top of the pipe, 22½ in. from the vertical. Good results were claimed. The glass fiber blankets are made and sold in quantity for insulation purposes, and therefore inexpensive. Further, the USDA Agricultural Research Laboratory in Brawley was experimenting with the same filter material, including the amount the material compressed when subjected to compression in place. The writer then initiated comparative laboratory experiments with the following relative results (on the basis of flow from treatment No. 1 being 100):

Treatment No. 1–A continuous blanket (strip 12 in. wide) under the tile, and a blanket over the tile wide enough to go around the tile and lap over the bottom blanket, flow 100.

Treatment No. 2–Doughnut. A ring of fiberglass mat placed in the tongue and groove joint of the tile. Best doughnut used, flow 11.

Treatment No. 3–A collar of a different type of glass fiber material designed as a sleeve collar over each joint. Best collar used, flow 16.

Treatment No. 4–An envelope of Coachella filter sand around joints, flow 40.

On the basis of this laboratory performance, a replicated cooperative field experiment was initiated on the Hugh Bennett property near Firebaugh in the San Joaquin Valley. The treatments are as follows:

Treatment No. 1–Six-in. concrete drain tile with sand filter. San Joaquin Valley pit run sand, 18 tons per 100 lineal feet.

Treatment No. 2–Perforated (4 in.) bituminized fiber pipe, with perforations 45 deg apart, placed up, and covered with a continuous glass fiber blanket 1 in. thick by 12 in. wide.

Treatment No. 3–Six-in. concrete drain tile with 1-in. thick glass fiber rings (doughnuts) inserted in the joints and a 2-ft wide by 1-in. thick continuous glass fiber mat placed over the top.

Treatment No. 4–Six-in. concrete drain tile placed on top of a continuous glass fiber blanket 4 ft wide and 1 in. thick which went across the bottom and up one side of the trench. A continuous blanket 2 ft wide and 1 in. thick was tacked to the other side of the trench, then a 2-ft wide blanket 1 in. thick was placed over the tile.

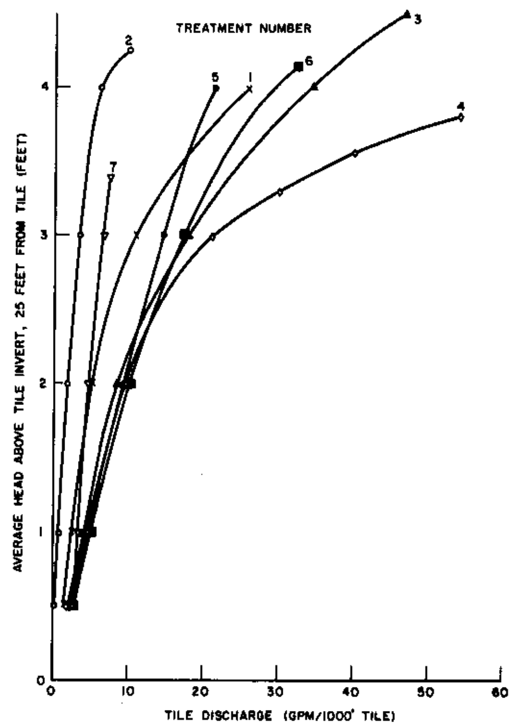

Figure 2. Relationship between tile discharge and average head above tile invert for the seven treatments.

Treatment No. 5–Perforated 4-in. bituminized fiber drainage pipe spirally wrapped with a glass fiber mat 1 in. thick. Perforations were the same as in treatment No. 2, but the perforations were placed down.

Treatment No. 6–Six-in. concrete drain tile with a 2-ft wide by 1-in. thick glass fiber mat placed over the tile only.

Treatment No. 7–Six-in. concrete drain tile with 2-ft wide by 1-in. thick glass fiber mats under and over the tile.

Treatments 4 and 7 were installed by hand. All other treatments were installed by machine. The machine used for treatments 1, 3 and 6 had a device for placing the tile, as it was shoved into position, under moderate longitudinal compression.

Yield of effluent from these tile lines has been in accordance with Figure 2. Treatment No. 2 is significantly lower than the other treatments, but within the other treatments differences are not significant. In June 1964, arrangements were made to dig clown to one tile line of each treatment. The results showed that there were obviously wider joint openings with hand-placed tile because considerable fine sand and silt was found in both treatments 4 and 7, and not in the machine placed concrete lines. This could have entered the pipe during "subbing-in."

A small amount of soil material was found in the perforated tile lines (treatments 2 and 5). Also, small cores of mat and soil were found in the tile with perforations down (treatment 5). No cores were found in the tile line with perforations up, but very little water had flowed into this line.

The glass fiber material appeared never to have been wet above about the centerline of the tile lines (above the elevation of the perforations for treatment 2). Below this level, the material was compressed to a thin sheet and was mushy and without resilience. The soluble binder had been leached away. The failure of this material may be the reason why significant differences between treatments were not observed. There is some thought that possibly a glass fiber material with very coarse strands of borosilicate glass and an insoluble binder might be suitable, but the material manufactured for insulation purposes is not. One reason for differences between field and laboratory tests might have been that the field trials were in a highly saline sodic soil; the laboratory work was not.

PROTECTION FROM BACKFILL

The usual practice is to backfill the trench Immediately behind the tile-laying machine. The loose material is mounded over the trench and then a furrow is constructed along its length. This furrow is then filled with water and ponded to "sub in" the backfill. Many experienced installers believe that it is during the subbing-in that much damage can occur. There is the belief that the trench is then a trough filled with water and loose soil, and that the resultant soupy soil, here and there, pipes through the filters or envelope. This could have been the case at the Bennett plots but not in Imperial Valley where the soil must have entered gradually over a period of years. Following subbing-in, most soil dries and achieves considerable structure–particles binding

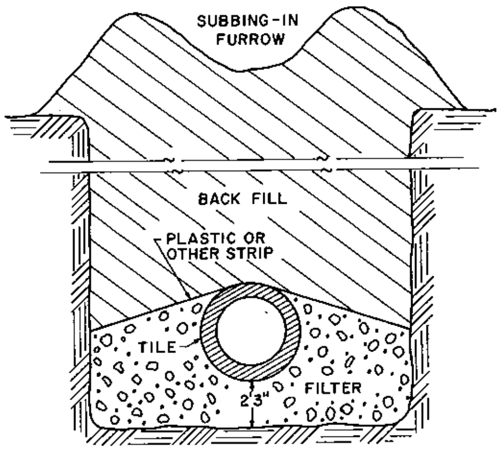

Figure 3. Profile of a tile installation with filter, demonstrating flow towards the tile when the filter above the water table is unsaturated and the interface between soil and filter acts as a barrier. This condition is normal, even when the soil is saturated above the tile.

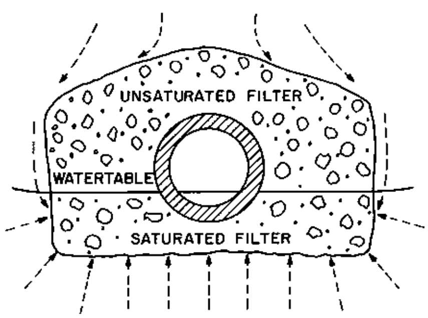

Figure 4. Profile of suggested installation technique, showing tile, filter, plastic or other type protective strip, and "subbing-in" furrow for compacting backfill.

together–and stability. The writer has simulated subbing-in in the laboratory, and has found that it can entrap considerable air which discharges into the tile in a pulsating manner with considerable disturbance and displacement of the filter. This verifies the theory of damage during sub bing-in.

Where envelopes or filters are used, it is usual practice to place the material all around the pipe. If not, it is placed only on top, possibly because that is the easiest and cheapest place to put it. Theory would indicate the area above the tile contributes little or nothing to flow into the tile. The lack of wetting of the glass fiber mat above about the centerline of the tile at the Bennett plots confirms this theory. Movement of water in soil below a water table is said to be saturated flow, and the water table represents the elevation of zero atmospheric pressure. Above a water table, soil water is under a negative pressure. It moves in response to hydraulic gradient, but it is sucked along due to the affinity of soil for water. With an abrupt change from a fine to a coarse textured material in the direction of movement, there is an effective break in the hydraulic gradient. This is because water moves through the fine capillaries of the fine textured soils at higher suction values than through the relatively extremely large capillaries of the coarse textured soil, and those capillaries are few and far between with the coarse textured soil.

This concept is shown in Figure 3, the cross section of a tile line with a gravel envelope. Presume that it is just before irrigation and there has been recession of the water table so that flow is low. The water table in the tile and essentially in the envelope is the surface of the water in the tile. The envelope above this is unsaturated. With irrigation, water will start percolating downward through soil, and will move around the envelope through soil to the water table. The water table will rise, increasing flow, but above the water table the envelope remains unsaturated and the boundary between the soil and the gravel remains an interface, blocking flow through it. The boundary between the soil and the gravel remains an effective interface until the pipe is flowing full and the water is under sufficient pressure to raise the water table to the top of the gravel. The tile would then not be functioning as a drain. Design criteria are such that the pipe should never run full.

Any function of envelope or filter material over the tile is to prevent the piping of flow directly to a joint or other opening, primarily during the subbing-in period and while the soil structure of the backfill is becoming stabilized. There is no objection to a barrier here being impervious, and It should have structural strength to span any joints or other openings into the pipe without failure. Asphalt roofing paper, strips of plastic sheeting, or other materials that adhere to these criteria can be used. The material should be wide enough to cover the exposed top of the pipe and the envelope to prevent piping around the edge (Fig. 4).

CONCLUSIONS

This has been intended as a summary of experiments with tile filters) or "subdrain filters" in highway engineer's terminology. Actually, it has gone beyond this in the hopes of developing a broader viewpoint as to the environment in which such facilities might be required, and information on what is required.

As to subdrain filters, a graded sand is the most satisfactory material that has been reasonably proven. The glass fiber mat material produced for insulation purposes is not satisfactory. There may be other types of glass fiber mat that are satisfactory, but this is not yet proven.

REFERENCES

1. Bush, A. F. Urban Atmospheric Pollution. Civil Engineering, pp. 66-68, May 1965.
2. Standard Specifications for Concrete Aggregates. ASTM Designation C33-64.
3. Terzaghi, K., and Peck, R. B. Soil Mechanics in Engineering Practice. John Wiley & Sons, New York, 1948.
4. Investigation of Filter Materials for Underdrains. U. S. Army Engineer Waterways Experiment Station, Tech. Memo. 183-1, Vicksburg, Miss., 1941.
5. Leatherwood, F. N., and Peterson, D. F., Jr. Hydraulic Head Loss at the Interface Between Uniform Sands of Different Sizes. Trans. Amer. Geophysical Union, Vol. 35, pp. 588-594, Aug. 1954.
6. Karpoff, K B. The Use of Laboratory Tests to Develop Design Criteria for Protective Filters. Earth Laboratory Branch, Engineering Laboratories Division, Bureau of Reclamation, USDI. Paper presented at the 58th Annual Meeting of ASTM, Atlantic City, New Jersey, June 27, 1955.
7. Qazi, A. R. Design Criteria for Tile Drain Filters. Master's Thesis, Colorado State Univ., 1961. 90 pp.
8. des Bouvrie, C. Design Criteria for Drain Tile Filters. Master's Thesis, Colorado State Univ., 1962. 132 pp.
9. Dixon, W. J., and Massey, F. J. Introduction to Statistical Analysis, 2nd Ed. McGraw-Hill, New York, 1957.

Laboratory Investigation of Soil Infiltration Through Pipe Joints

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This paper describes an investigation of the infiltration characteristics of four soils–a poorly graded medium to fine sand (SP), a uniformly graded fine sand (SP), a silt (ML), and a lean clay (CL)–in an effort to develop a system of classifying soils according to the degree that infiltration through pipe-joint openings may be expected to occur. The investigation included the design and construction of a model simulating a prototype pipe joint, a study of the feasibility of using the model for such studies, an investigation of the variables affecting soil infiltration, and an investigation of the infiltration of the four soils described.

● PROBLEMS associated with leaking joints and infiltration of soil into buried drains have been recognized for many years in certain regions of the United States. The seriousness of leaking joints in storm-drainage lines at airfields was noted in 1951 at Hunter Air Force Base (AFB), Savannah, Georgia. Field inspections of the storm-drainage system there revealed that pavements had failed in at least two locations as a result of loss of subgrade support due to infiltration of backfill and subgrade soils into the underlying storm-drainage pipe. Further inspection indicated that about one-fourth of all pipe joints in the drainage system, both concrete and metal pipe, were leaking, and large amounts of sandy subgrade soils had infiltrated the underlying drainage pipe in numerous places. An investigation by the U. S. Army Engineer District, Savannah, was made to obtain basic data for use in improving and revising existing Corps of Engineers criteria for design and construction of drainage systems. A report (1), published in 1955, included recommended material specifications and construdion procedures for providing watertight joints in both rigid and flexible drainage pipes.

During 1955, the U. S. Army Engineer Waterways Experiment Station (WES) conducted a series of field inspections of storm-drainage systems at various military installations in the South Atlantic and Gulf Coast areas (2). Numerous localized pavement failures were found which could be traced to loss of subgrade support due to infiltration of subgrade soil into both concrete and corrugated-metal drainage pipes. Similar serious infiltration was encountered at other locations, particularly at Plattsburgh AFB, New York. Material specifications and construction procedures for watertight joints were first included in the Corps of Engineers guide specification for storm-drainage systems (3) published in 1958. After the publication of this guide specification, requirements for watertight joints were included in all military construction specifications for projects where objectionable infiltration of soil was likely. Because these requirements are not economically justified where watertight joints are not actually needed, criteria are needed to guide the designer in determining where, if at all, watertight joints are required at a particular construction site.

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The purpose of the study was to determine the susceptibility of soils to passing through pipe joint or seam openings, with primary emphasis on developing a system of classifying soils according to their infiltration probabilities. This was to be accomplished by developing relations between the occurrence of significant infiltration of various soil types and certain variables believed to affect soil infiltration. This would provide guidance in determining when watertight joints should be specified in particular soils. The requirement for tight joints is related to the characteristics of the surrounding soil, position of the water table, and the hydrostatic pressure tending to move the soil particles through the joints. The basic test apparatus used in this study was designed to investigate these conditions. Although the test apparatus simulated a joint opening, the results of the tests are applicable to other types of pipe openings, such as cracks in rigid pipe.

The initial tests to determine the suitability of the model for studying soil infiltration through pipe joint or seam openings and to evaluate the effects of some of the more important variables believed to control soil infiltration were accomplished by the U. S. Army Engineer Ohio River Division Laboratories (ORDL). Subsequently, the principal program of tests was accomplished at WES.

DEFINITIONS

The following terms pertaining to the model and infiltration are used throughout this paper and are defined here for convenience:

Datum plane. Reference point for all waterhead measurements. The datum plane has been taken as the bottom elevation of the soil specimen in the model which coincides with the bottom elevation of the simulated pipe-joint opening.

Waterhead. Total head in feet of water applied to the soil specimen in the infiltration tests. The head was measured either from the datum plane to the top elevation of the water above the soil specimen in the case of low heads, or by a pressure gage located above the soil specimen, with readings corrected to the datum plane.

Joint opening. Vertical distance, in inches, between the upper and lower surfaces of the simulated corrugated joints.

Flow. Quantity of water percolating through the soil mass and discharging through the individual simulated pipe-joint openings, expressed in cubic centimeters per minute.

Infiltrated material. Soil carried through the simulated joint openings of the model by the flow of water.

Total washout. Infiltration of material to the extent that an open channel developed through the soil specimen.

Partial washout. Condition in which infiltration of material caused a depression to be formed in the surface of the soil specimen; however, no open channel developed and no further significant soil infiltration occurred thereafter.

TEST PROGRAM

Soils Tested

Four soils, two sands, a silt, and a lean clay were used in the tests. These soils were selected to provide ranges in gradation and plasticity similar to those in the field where infiltration problems have been experienced.

One sand was a medium to fine sand (hereafter termed M-F sand), laboratory blended to approximate the gradation of the sands in the area of Plattsburgh AFB, New York, where extensive infiltration into drainage pipes has occurred. The other sand was a uniform fine sand (hereafter termed uniform sand), a natural material obtained from Hurlburt Air Force Auxiliary Airfield, Florida, during a field investigation of soil infiltration into drainage pipes beneath parking aprons. The silt and lean clay were natural materials obtained from the Vicksburg, Mississippi, area. Because of its low plasticity, the silt is susceptible to infiltration through small openings; the lean clay was selected to represent a borderline material with respect to susceptibility to infiltration.

TABLE 1
SUMMARY OF LABORATORY TEST RESULTS

Material	Specific Gravity (G_s)	CE-55 Compaction: Maximum Dry Density (pcf)	CE-55 Compaction: Optimum Water Content (%)	Atterburg Limits: LL	Atterburg Limits: PL	Atterburg Limits: PI
Medium to fine sand (SP)	2.70	112.8	9.0		Nonplastic	
Uniform fine sand (SP)	2.70	107.6	14.0		Nonplastic	
Silt (ML)	2.72	112.9	13.2	31	24	7
Lean clay (CL)	2.67	116.3	12.8	36	23	13

The results of the specific gravity, Atterberg limits, and compaction tests on the four soils are summarized in Table 1. The compaction tests were performed using the test procedure outlined in MIL-STD-621A, Dec. 22, 1964, Test Method 100, Compaction Effort Designation CE-55. The CE-55 compaction method is nearly identical with the modified AASHO method. In this report, maximum density obtained by the CE-55 method is referred to as the CE-55 maximum dry density, and any other density is referred to as percent CE-55 maximum dry density. Gradation curves of the soils are shown in Figure 1. Compaction curves of the four soils are shown in Figure 2. The compacted densities of the infiltration test specimens are also shown in Figure 2.

Test Apparatus

The test apparatus for the infiltration studies was designed and constructed at WES. Since a study of infiltration through slotted relief well screens had been made previously at WES (4), it appeared desirable to provide means for comparing infiltration characteristics of various soils using both simple slots and simulated externally banded corrugated-metal pipe joints. The test apparatus was a 6- by 12- by 26-in. high plastic container on a fiber-glass base that modeled two different lengths of corrugated-metal pipe joints with provision for varying the joint openings with metal spacers; the base also contained a slot, the opening of which could also be varied. Views of the plastic container and fiber-glass base are shown in Figures 3 and 4 and a diagram of the apparatus used for the tests is shown in Figure 5.

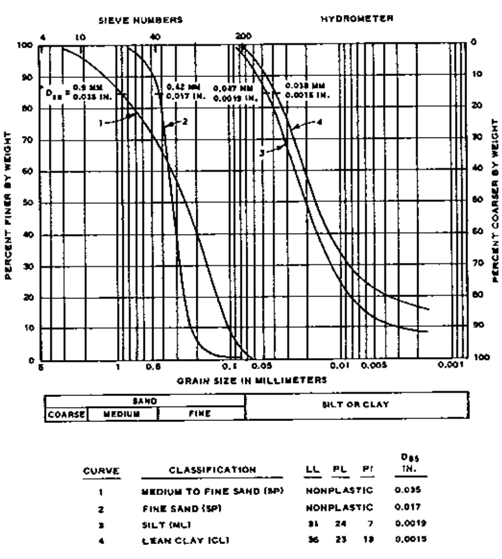

Figure 1. Gradation curves and classification data.

Connections for piezometer tubes were provided at various vertical heights in one side of the plastic box to permit piezometric head measurements in the soil specimen (Fig. 5). The setup for application of low heads (less than 4.26 ft) and high heads (4.26 to 23.79 ft) is shown in Figures 6 and 7, respectively. Heads up to and including 4.26 ft were applied and maintained at a constant level by means of overflow outlets located in the sides of the model (Fig. 6). The inflow of water to the model was regulated by means of a valve to equal the combined flows through the specimen and the overflow outlet. For waterheads greater than 4.26 ft, the standpipe was removed, and a flat plastic top with a rubber gasket was placed on the model and bolted to provide an airtight seal (Fig. 7); the pressure head was then applied using water pressure controlled by a pressure regulator. Figures 6 and 7 show that pressure measurements were taken using piezometer tubes for low heads and using pressure gages for high heads.

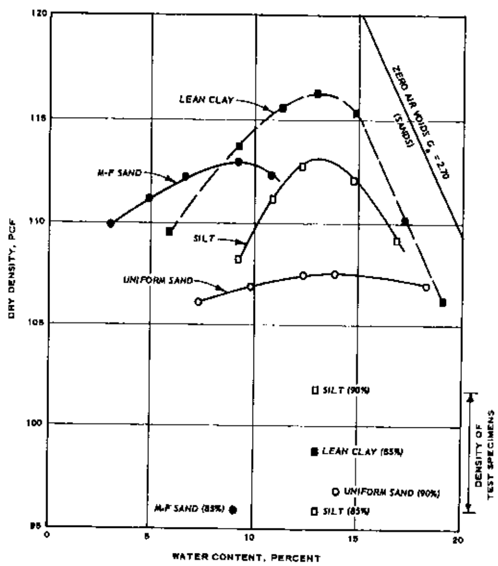

Figure 2. Compaction curves and densities of test specimens.

One of the joints in the base was 4 in. long, representing one-half a standard joint in which an 8-in. wide connecting band is used to join two sections of corrugated pipe. The other joint was 6 in. long, representing one-half a 12-in. wide connecting band. These simulated prototype joints were formed by casting fiber glass on a section of 16-gage, standard corrugated-metal culvert-pipe sheet. Each joint was composed of two castings, one casting representing the pipe and the other representing the connecting band.

Preparation of Test Specimens

Soil at optimum water content as determined by the CE-55 compaction method was placed in the test container in layers and hand-tamped with a wood rammer to the desired density. In the ORDL tests, the material was placed in four equal layers; in the WES tests, the number of layers was increased to six in an attempt to obtain, a more uniform density throughout the specimen. With several exceptions, all specimens were 0.2 cu. ft in volume,

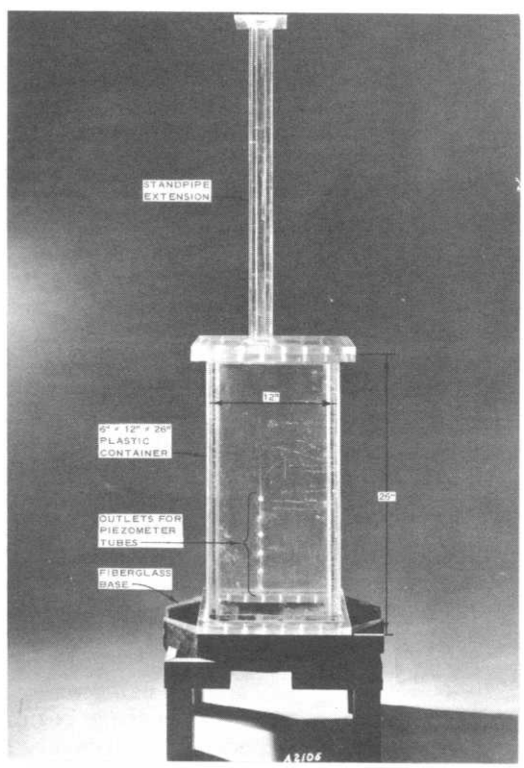

Figure 3. Assembled test apparatus.

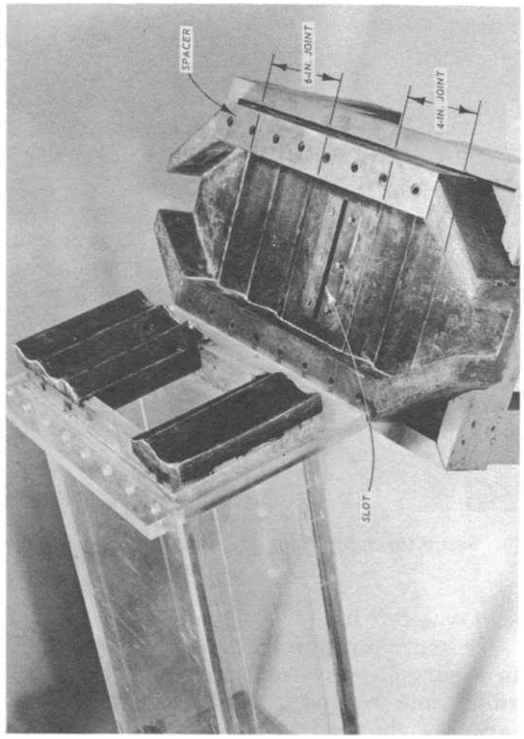

Figure 4. Disassembled test apparatus.

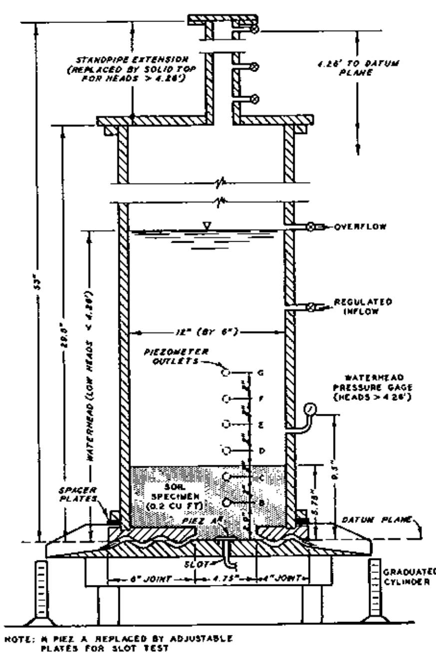

Figure 5. Infiltration apparatus.

resulting in a specimen with a height of approximately 5.75 in. above the datum plane (Fig. 5). Immediately following compaction, the soil specimen was saturated from the bottom upward by placing the test container and assembled fiberglass base in a reservoir of de-aired water and subjecting them to a head of approximately 7 in. A surcharge approximately equal to the head of water was placed on the surface of the soil specimen in the model to prevent an increase of volume during saturation. The surcharge was distributed over the surface of the specimen by weights placed on the honeycombed or perforated plastic insert shown in Figure 6. The specimen was considered saturated when the level of water above the soil specimen in the model was the same as the level of water in the reservoir. The time required for saturation varied, depending on the soil type and soil density.

At the end of the saturation period, after the joint openings were sealed off, the surcharge was removed, and the model was removed from the saturation reservoir and prepared for application of the desired

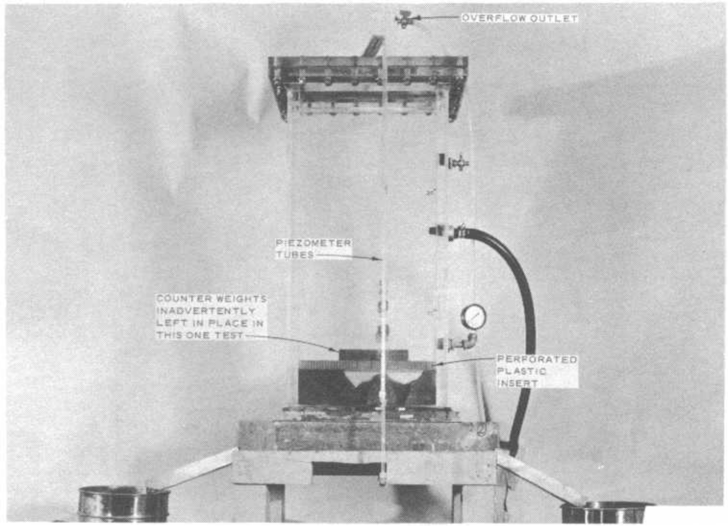

Figure 6. Setup for application of low waterheads (after test).

heads. The plastic insert was left in place to protect the surface of the specimen from scour during the test. (The surcharge weights had been inadvertently left on the insert during the test shown in Figure 6. However, upon disassembly of the apparatus, it was found that the insert had become wedged in the apparatus and, consequently, it is not believed that the surcharge weights had any measurable effect on the test results.) In the ORDL tests, new soil specimens were compacted for each test with a different head or joint opening, with only a few exceptions. In the WES tests, the same soil specimen was subjected to increasing heads until total washout of material occurred or the maximum test head (23.8 ft) was reached. In the ORDL tests, de-aired water was used for saturating specimens and in all infiltration tests involving heads less than 5 ft; tap water was used in all WES tests and in all ORDL tests involving heads higher than 5 ft.

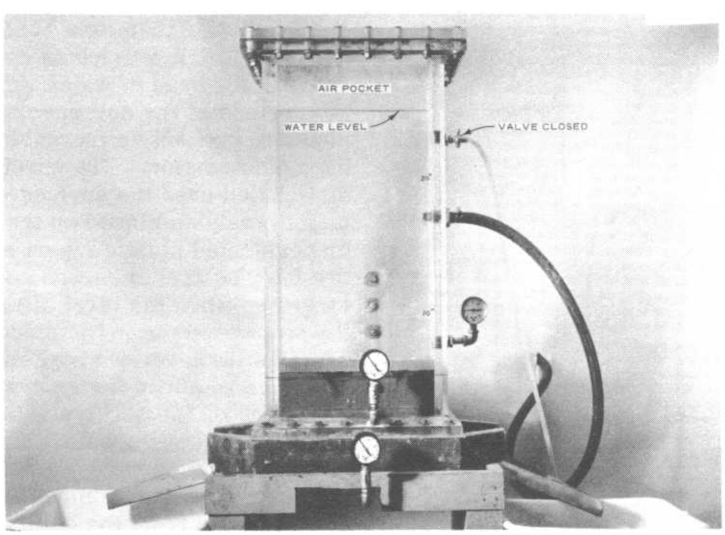

Figure 7. Setup for application of high waterheads (after test).

Test Procedure

Reference was made to a previous study (5) to obtain general qualitative data on the sizes of joint openings which might be expected when sections of corrugated-metal pipe are joined with coupling bands in the field. These data were used as a basis for selecting joint openings of 0.007 to 0.260 in. for the laboratory infiltration tests.

In the infiltration tests on the corrugated joints, the joint opening was varied by using metal spacers of known thickness between the plastic box and fiber-glass base. Because of irregularities in the corrugations of the pipe used in casting the model, the actual opening in the tests did not correspond to the thickness of the spacer. In the ORDL tests, the actual openings were determined by measuring changes in thickness of two hollow-core solder wires placed in the corrugations prior to assembling the model, and then bolting the model together using a spacer of known thickness. At WES, the procedure used was to place liquid body solder in the model joints and assemble the model using a given spacer. After disassembly of the model, the hardened sheet of formed body solder was cut into strips (one strip for each ridge and valley); the thickness of the strips was determined with calipers. There was considerable variation in joint opening in both longitudinal and transverse directions relative to the direction of flow. It is expected that variations of the same order would be found in any similar pipe used in construction projects.

Because the interest of this study is in soil infiltration through the joint and not in flow, the size of the joint opening reported in the test results for a given spacer is that of the largest opening found in that transverse cross section of the joint which had the smallest joint opening in the longitudinal direction. This reported value is considered to be the controlling opening with respect to the maximum-size particle which might pass through the joint.

In the ORDL tests, the desired waterhead was maintained on the soil specimen for 3 to 8 hr (except for some special tests). In the WES tests, initial pressure heads were maintained about 20 hr, and subsequent heads were maintained at least until flow conditions had stabilized. In all tests on the corrugated joints, the water flowing through the soil specimen passed simultaneously through both the 4- and 6-in. long openings, with the slot opening sealed off. In the WES tests using the slot, the corrugated openings were sealed off.

In the ORDL tests, the discharge from each corrugated opening was passed through a 200-mesh screen and filter paper to collect any material discharged for quantity and gradation determinations. In the subsequent WES tests, no measurements of infiltrated material were made, but visual estimates were made. Flow measurements were made periodically by measuring the amount of water discharged through each opening. In the ORDL tests, piezometer observations were made on 0.2-cu ft specimens at locations B and C (Fig. 5). In the WES tests, piezometer observations were made at location A in the base of the model and at location B on the side of the soil specimen (Fig. 5).

ORDL TESTS

The first tests with the infiltration model were performed by ORDL using M-F sand and silt. Detailed data on the ORDL tests are given in the unpublished ORDL interim report on the study (6).

Tests conducted at ORDL can be grouped into three categories (Table 2). Discussion of the results of categories A and B tests is included with discussion of results of the WES tests. Table 3 summarizes pertinent results of the exploratory infiltration tests on medium to fine sand and on silt. The category C tests were performed to make limited evaluations of the effects of test duration, specimen height, high heads, density, length of joint, and vibration. The following general conclusions were drawn from the category C tests[1] for guidance in the conduct of subsequent tests:

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[1]It should be noted here that measurements of simultaneous water flow and infiltration through the two joints of different lengths did not provide data amenable to rigorous analysis; this is also true of similar data observed in tests involving different specimens. Local nonuniformity in density, particularly

TABLE 2
VARIABLES STUDIED IN ORDL TESTS

Variables	Test Category and Soil Type			
	A	B	C	
	M-F Sand	Silt	M-F Sand	Silt
Density, % max CE-55	85	85	85	85-90
Specimen size, cu ft	0.2	0.2	0.2-0.5	0.2-0.5
Test heads, ft	1-5	6.5	1.0-24.6	6.5-26.5
Corrugated joint openings, in.	0.007-0.210	0.007-0.158	0.013-0.210	0.007-0.105
Test time, hr	To 8.0	To 6.8	1.5-39.5	5.5-42.0
Number of tests	24	6	8	4
Summary of results	Table 3	Table 4	Table 5	Table 5

TABLE 3
SUMMARY OF ORDL INFILTRATION TESTS

Test No.	Joint Opening (in.)	Water-head (ft)	Test Period (hr)	Avg. Flow (cc/min) 4-in. Joint	Avg. Flow (cc/min) 6-in. Joint	Infiltration Material (g) 4-in. Joint	Infiltration Material (g) 6-in. Joint	Remarks
Category A: Medium to Fine Sand at 85% Maximum Density								
1[a]	0.210	1.02	2.5	–	–	–	–	Total washout
2[a]	0.158	1.01	6.0	63.1	124.5	305.9	0.02	Partial washout
3[a]	0.158	2.02	0.2	–	–	629.9	–	Total washout
4[a]	0.105	1.01	6.0	18.5	81.9	0	0	
	0.105	2.01	2.0	138.6	183.9	32.9	36.8	
	0.105	3.01	1.0	489.5	274.7	342.9	105.4	Partial washout[b]
	0.105	4.01	8.0	362.8	454.0	53.0	264.5	Partial washout[b]
5	0.053	1.01	6.5	106.4	151.9	0.9	0.4	
	0.053	2.01	7.0	454.9	311.6	29.4	10.4	
6	0.053	3.01	7.0	393.8	297.8	63.3	19.5	
7[a]	0.053	4.01	5.5	585.6	429.5	35.5	12.0	
8	0.026	1.00	5.5	129.7	98.1	0.1	0.1	
	0.026	2.00	6.0	477.0	260.3	7.5	0.1	
9	0.026	3.00	6.0	395.2	312.5	13.7	1.8	
10[a]	0.026	4.00	6.0	757.8	304.5	10.0	3.0	
11	0.013	1.00	6.5	172.3	76.0	0.2	0.1	
	0.013	2.00	7.0	380.7	249.4	1.5	0.9	
12	0.013	3.00	7.0	360.3	280.5	3.9	2.8	
13[a]	0.013	4.00	6.5	592.4	674.8	1.9	3.1	
14[a]	0.007	1.00	7.0	88.5	73.8	0.1	0.1	
	0.007	2.00	6.0	190.8	208.1	0.3	1.2	
	0.007	3.00	5.5	248.9	537.1	0.3	2.5	
	0.007	4.00	3.5	304.3	725.4	0.3	3.0	
	0.007	5.00	3.0	309.0	869.0	0.4	3.0	
Category B: Silt at 85% Maximum Density								
15	0.158	6.50	–	–	–	–	–	Total washout[c]
16	0.105	6.50	5.50	1.8	5.4	34.1	110.8	Partial washout[d]
17	0.053	6.50	4.75	7.9	8.9	8.1	8.0	Total washout[e]
18	0.026	6.50	6.75	5.2	6.1	731.3	53.8	Partial washout[f]
19	0.013	6.50	4.75	13.0	5.9	–	–	Partial washout[g]
20	0.007	6.50	6.75	10.4	6.7	31.0	45.0	

[a] Data used in Figure 8.
[b] Partial washout occurred soon after waterhead was applied. Test continued with only small amount of additional infiltration.
[c] Total washout occurred before total waterhead could be applied.
[d] Partial washout occurred quickly after application of waterhead; however, infiltration slowed and test was continued for 5.5 hr.
[e] Total washout occurred after 4.75 hr of test. Flow and infiltrated material values shown were taken after 4.25 hr of test.
[f] Partial washout occurred through 4-in. joint after 1.75 hr of test. Valves shown are for total time of test (6.75 hr).
[g] Partial washout occurred through both joints after 1.75 hr of test. Flow measurement was made immediately prior to partial washout. Infiltration slowed and test was continued for 4.75 hr.

adjacent to the openings, and channelization of water flow through the soil specimen were probably the variables complicating analysis of the frequently inconsistent results from which, however, significant general trends were indicated.

1. Duration of tests: Practically all of the soil infiltration occurred in the early stage of each test under a given head. For example, 96 percent of the soil infiltration in test 21 (Table 4) occurred in the first 6½ hr. Generally, 80 to 90 percent occurred within a few minutes after starting of the flow. Furthermore, if washouts occurred, they did so within the first 5 to 6 hr. Therefore, it was not necessary to continue subsequent tests over longer periods of time.

2. Height of specimen: Comparative tests were performed on specimens having heights of 13 in., volume = 0.5 cu ft, and the normal heights of 5.75 in., volume = 0.2 cu ft (Table 4, tests 22, 23, 29, and 30). Under the same head, the shorter specimen was, of course, more susceptible to soil infiltration and washout because of the larger hydraulic gradient. When the head was increased approximately twofold on the 13-in. specimen (test 23), making the hydraulic gradient fairly close to that producing the total washout of the 5.75-in. specimen (test 22), a partial washout occurred. Thus, it appeared that the 0.2-cu ft specimen size was satisfactory for further tests.

3. High heads: As was true with the infiltration tests using lower heads, the greatest proportion of infiltrated material was produced in the early stage of the tests. With each increase in head, additional material infiltrated, but unless a partial or total washout occurred under the increased head, little more and sometimes less material infiltrated than under previous lower heads (test 28, Table 4).

4. Density: Single comparative tests were performed using silt, with one specimen at 85 percent CE-55 maximum dry density and the other at 90 percent (tests 29 and 31, Table 4). With all other test conditions constant, considerable infiltration occurred in the test on the lower density specimen, while none occurred with the higher density specimen.

5. Length of joint: In the tests on sand specimens, there was a tendency for less material to infiltrate through the 6-in. joint than through the 4-in. joint, although the reverse sometimes occurred. The 6-in. joint provided more corrugations, and the settling out of coarser particles in the valleys of the corrugations formed a filter to prevent further infiltration. The amount of infiltration through the two joints in the tests of silt specimens apparently was not affected by the joint length.

6. Vibration: The few observations in which specimens of sand and silt were vibrated by various means after the conclusion of the regular tests indicated the importance of vibration relative to magnitude of soil infiltration. Some additional sand infiltrated each time the model was struck or otherwise vibrated; vibration had a somewhat lesser effect on the silt specimen.

WES TESTS

The ORDL tests using the infiltration model showed the importance of soil type, soil density, head of water, joint opening, and vibration with respect to soil infiltration through pipe joints. The WES testing program extended the testing on the two soils (M-F sand and silt) tested by ORDL, and included testing of the two additional soils (uniform sand and lean clay).

Each soil specimen was subjected to a series of waterheads, beginning at a relatively low head and increasing the head in increments until either a washout occurred or a maximum head of 23.79 ft was attained. The test duration at each head was varied, the head being held constant until either a washout occurred or the pressures measured by the piezometer in the base of the model had become relatively stable. In some instances, the head was maintained for longer periods of time.

The investigation of the effect of joint configuration on soil infiltration was confined to a comparison of infiltration through the two lengths of corrugated openings with that through a simple slot opening. Investigations of the 4- and 6-in. lengths of corrugated openings were conducted simultaneously, while investigation of the simple slot opening was conducted separately.

Although piezometer observations and measurements of flow through the openings were made in the WES tests, these data are not presented, as all attempts to relate these data to size of joint openings or to head of water proved fruitless because of inconsistent variations in the values. The reasons for the variations are not known, but

TABLE 4
SUMMARY OF ORDL INFILTRATION TESTS

Test No.	Size of Soil Specimen (cu ft)	Joint Opening (in.)	Waterhead (ft)	Test Period (hr)	Average Flow (cc/min)		Infiltrated Material (g)		Purpose of Test	Remarks
					4-in. Joint	6-in. Joint	4-in. Joint	6-in. Joint		
Category C: Medium to Fine Sand at 85% Maximum Density										
21	0.2	0.053	4.00	6.50	652	218	54.30	8.1	Investigate time effects	Vibrated at end
			4.00	8.00	1,518	173	0.60	0.8		of test[a]
			4.00	8.00	1,137	115	0.20	0.1		
			4.00	8.00	799	66	0.10	0.0		
			4.00	8.00	659	57	0.50	0.1		
			4.00	1.00	541	48	0.00	0.0		
				39.50			55.70	9.1		
22	0.2	0.210	1.02	2.50	–	–	–	–	Study effects of soil	Total washout
23	0.5	0.210	1.02	1.50	57	108	<0.10	<0.1	specimen height	Partial washout
			2.00	1.50	121	170	–	–		
24	0.2	0.105	24.58	4.00	4,050	1,273	148.00	126.8	Study effects of very	Vibrated at end
									high waterheads	of test[b]
25	0.2	0.053	24.58	1.00	14,276	1,114	Not measured			
26	0.2	0.026	24.58	6.00	1,435	403	16.70	3.1		
27	0.2	0.013	24.58	6.00	4,125	532	6.30	0.6		
Category C: Silt at 85% Maximum Density (Except Test 31)										
28	0.2	0.007	6.50	6.00	10	7	31.0	45.0	Study effects of time	Vibrated at end
			11.50	6.50	6	6	29.0	68.0	and high head	of test[c]
			16.50	6.75	10	4	36.0	13.0		
			26.50	6.75	7	15	21.0	10.0		
			26.50	4.50	14	14	1.0	1.0		
			26.50	6.50	14	13	0.0	0.0		
			26.50	5.00	16	9	0.0	0.0		
				42.00			118.0	137.0		
29	0.2	0.105	6.50	5.50	2	5	34.1	111.0	Study effects of	
30	0.5	0.105	7.00	6.50	5	1	7.4	24.0	specimen height	
29	0.2	0.105	6.50	5.50	2	5	34.1	111.0	Study effects of soil	
									density	
31	0.2	0.105	6.50	6.50	4	1	0.0	0.0		Test 31 was at
										90% density

[a]Model was vibrated with audio vibrator and by tapping with screwdriver. Additional slight infiltration occurred as long or model was vibrated.
[b]Vibrated by striking with screwdriver. Additional infiltration observed but ceased rapidly when tapping was stopped.
[c]Model was vibrated with mechanical vibrator for 40 min. At beginning of vibration additional infiltration was observed but it ceased quickly.

TABLE 5
INDEX OF WES INFILTRATION TESTS

Joint Opening (in.)	M-F Sand		Uniform Sand	Silt			Lean Clay
	CJ[a] (85%)[b] Table 7	Slot (85%) Table 8	CJ (90%) Table 9	CJ (85%) Table 10	CJ (90%) Table 11	Slot (85%) Table 12	CJ (85%) Table 13
0.013				1			
0.026			1	2			
0.050						16	
0.053						17	
0.054			2,3	3			
0.080	1[c]		4,5,6				
0.095		7,8,9				18	
0.105	2			4,5			
0.110		10					
0.118		11					
0.124						19	
0.125		12				20	
0.126	3,4		7,8				
0.130				6			
0.150		13					
0.158	5			7,8	11		
0.188						21	
0.209					12,13		
0.210	6			9			1,2
0.250						22	
0.260				10	14,15		3

[a]CJ = corrugated joint.
[b]Percent of maximum dry density.
[c]Test numbers.

are thought to be associated with nonuniformity of soil density within the test specimen.

Table 5 is an index of the WES tests by test number, indicating soil tested, joint openings and type of opening (corrugated joint or slot), together with references to the tables summarizing the observational data of the tests (Tables 6-9).

Thirteen tests were performed using M-F sand specimens compacted to 85 percent CE-55 maximum dry density. Of these, 6 were performed with corrugated joint openings ranging from 0.080 to 0.210 in. and 7 were performed with slot openings ranging from 0.095 to 0.150 in.

Eight tests were performed on the uniform sand compacted to 90 percent CE-55 maximum dry density, with corrugated openings ranging from 0.026 to 0.126 in. (It was planned to place the sand at a density corresponding to 85 percent of the CE-55 maximum density. On saturation of the first sample, it was found that the density increased to about 90 percent CE-55 maximum dry density, and therefore it was decided to test all specimens at that density.)

Twenty-two tests were performed on the silt, 15 with corrugated joint openings from 0.013 to 0.260 in. and 7 with slot openings from 0.050 to 0.250 in. In the tests with corrugated openings, 10 specimens were compacted to 85 percent CE-55 maximum dry density and 5 specimens to 90 percent CE-55 maximum dry density. The specimens for the 7 slot tests were all compacted to 85 percent CE-55 maximum dry density.

TABLE 6
SUMMARY OF INFILTRATION TEST DATA

Specimen No.	Joint Opening (in.)	Test Duration (hr)	Test Head (ft)	Remarks
Corrugated Joint Opening, Medium to Fine Sand at 85% Maximum Dry Density				
1	0.080	23.0	1.97	A small amount of soil infiltration generally occurred for a short period of time with each change in head. No washout occurred
		23.5	3.02	
		23.5	4.26	
		8.0	5.39	
		3.0	6.54	
		4.0	7.69	
		4.3	8.84	
		3.0	9.99	
		4.0	12.29	
		3.5	14.59	
		1.0	16.89	
		3.2	19.19	
		3.2	23.79	
2	0.105	22.0	1.97	No infiltration occurred until application of the 4.26-ft head, then only a small amount for a short period of time. Partial washouts occurred through the 6-in. joint 5 min after application of 6.54-ft head and then through the 4-in. joint after 1 hr of testing at this head
		24.0	3.02	
		24.0	4.26	
		7.5	6.54	
3	0.126	21.0	1.98	A partial washout occurred through the 6-in. joint 30 min after application of the initial head (1.98 ft). The test was continued. A small amount of infiltration generally occurred for a short period of time with each change in head. No further washouts occurred and test was stopped at end of work week
		23.5	3.01	
		23.7	3.51	
		96.0	4.26	
		4.0	5.39	
		2.2	6.54	
		8.0	7.69	
		4.0	8.84	
		19.0	9.99	
		4.0	12.29	
		1.0	14.59	
4	0.126	71.0	1.98	A moderate amount of infiltration occurred through the 4-in. joint at the initial head (1.98 ft). No further infiltration occurred until the head was being raised from 7.69 ft. A total washout occurred through both joints at a head of 9.50 ft
		23.5	3.01	
		23.5	3.51	
		23.5	4.27	
		4.0	5.39	
		3.5	6.54	
		2.0	7.69	
		–	9.50	
5	0.158	21.5	1.05	A total washout occurred through the 4-in. joint at a head of 1.58 ft while the head was being raised
		–	1.58	
6	0.210	1.0	0.67	A total washout occurred through the 4-in joint 5.5 hr after application of the 1.07-ft head
		5.5	1.07	

TABLE 6 (Continued)

Specimen No.	Joint Opening (in.)	Test Duration (hr)	Test Head (ft)	Remarks
Simple Slot Opening, Medium To Fine Sand at 85% Maximum Dry Density				
7	0.095	17.0	1.98	No infiltration occurred. Test was discontinued after a leak developed in the model
		23.5	3.02	
		0.6	4.27	
8	0.095	21.5	1.98	A small amount of infiltration occurred with application of the initial head (1.98 ft). No further infiltration occurred and test stopped at head of 14.59 ft because large flow of water involved no infiltration
		4.2	3.04	
		18.0	4.26	
		7.0	6.54	
		4.0	7.69	
		3.0	8.84	
		4.6	9.99	
		3.2	11.14	
		4.6	12.29	
		2.8	13.44	
		3.0	14.59	
9	0.095	3.2	1.98	A small amount of infiltration occurred during the first two heads (1.98 and 3.01 ft). No further infiltration occurred
		2.8	3.01	
		4.5	4.26	
		2.8	6.54	
		4.5	7.69	
		3.2	8.84	
		3.2	9.99	
		2.2	12.29	
		2.0	14.59	
		1.2	16.89	
		2.0	19.19	
		2.5	21.49	
		2.2	23.79	
10	0.110	3.0	1.98	A small amount of infiltration occurred for short periods of time during each head up to and including the 9.99-ft head. No further infiltration occurred
		19.0	3.02	
		24.0	4.26	
		3.2	6.54	
		3.0	7.69	
		4.2	8.84	
		3.0	9.99	
		8.0	12.29	
		4.6	14.59	
		3.0	16.89	
		8.0	19.19	
		3.3	21.49	
		4.2	23.79	
11	0.118	22.0	1.08	A small amount of infiltration occurred for short periods of time during each head up to and including the 6.54-ft head. No further infiltration occurred
		3.5	1.98	
		19.0	3.02	
		4.0	4.26	
		3.0	6.54	
		4.5	7.69	
		3.2	8.84	
		4.5	9.99	
		3.0	12.29	
		2.7	14.59	
		1.6	16.89	
		1.0	19.19	
		1.5	21.49	
		1.0	23.79	
12	0.125	–	0.50	A total washout occurred immediately on application of the initial head (0.5 ft)
13	0.150	–	0.00	A total washout occurred during saturation

TABLE 7
SUMMARY OF INFILTRATION TEST DATA
(Corrugated Joint Opening, Uniformly Graded Fine Sand at 90 Percent Maximum Dry Density)

Specimen No.	Joint Opening (in.)	Test Duration (hr)	Test Head (ft)	Remarks
1	0.026	2.7	1.98	A small amount of infiltration occurred sporadically
		20.0	3.01	during all heads up to and including that of 12.29 ft.
		3.0	4.26	No infiltration occurred at subsequent heads
		3.0	5.39	
		4.0	6.54	
		3.4	7.69	
		3.0	9.99	
		3.7	12.29	
		4.0	14.59	
		3.5	16.89	
		4.0	19.19	
		4.0	21.49	
		4.0	23.74	
2	0.054	21.0	1.98	A small amount of infiltration occurred for a short
		3.0	3.01	period on application of the 1.98- and 3.01-ft heads.
		17.5	4.26	A partial washout occurred through both joints 4 hr
		3.0	5.39	after application of the 4.26-ft head. Test con-
		3.5	6.54	tinued with amounts of infiltration occurring
		4.0	7.69	sporadically
		3.7	9.99	
		5.0	12.29	
		3.0	14.59	
		3.0	16.89	
		3.5	19.19	
		4.0	21.49	
		3.5	23.79	
3	0.054	3.5	0.67	Small to moderate amounts of infiltration occurred
		19.0	1.08	for short periods of time on application of each
		24.0	1.48	head. A partial washout occurred through both
		24.0	1.98	joints 4 hr after application of the 4.26-ft head.
		23.0	3.01	The test was continued with small amounts of in-
		4.0	3.51	filtration occurring for short periods on applica-
		19.0	4.26	tion of each subsequent head. Test stopped be-
		4.0	5.39	cause of very large flow of water
		3.7	6.54	
		8.0	7.69	
4	0.080	3.0	0.67	Moderate amounts of infiltration occurred for short
		19.0	1.07	periods of time on application of each head. A
		4.0	1.48	partial washout occurred through both joints 15 min
		0.2	1.98	after application of the 1.98-ft head. Test stopped
				because of very large flow of water
5	0.080	19.0	0.67	Moderate amounts of infiltration occurred for short
		4.0	1.08	periods of time on application of each head. A
		19.0	1.48	total washout occurred through both joints 15 min
		4.0	1.99	after application of the 3.01-ft head (Counter-
		0.4	3.01	weights inadvertently left in place in this one test.)
6	0.080	15.5	0.67	Moderate amounts of infiltration occurred for short
		4.0	1.08	periods of time on application of each head. A
		19.0	1.48	total washout occurred through the 4-in. joint at a
		5.5	1.98	head of 2.91 ft while the head was being raised
		–	2.91	
7	0.126	2.5	0.67	Moderate amounts of infiltration occurred for short
		17.0	1.08	periods of time on application of each head. A total
		0.2	1.48	washout occurred through the 4-in. joint 15 min
				after application of the 1.48-ft head
8	0.126	3.2	0.67	Moderate amounts of infiltration occurred for short
		19.0	1.07	periods of time on application of each head. A
		4.0	1.48	total washout occurred through the 4-in. joint
		–	1.70	immediately after application of the 1.70-ft head

TABLE 8
SUMMARY OF INFILTRATION TEST DATA

Specimen No.	Joint Opening (in.)	Test Duration (hr)	Test Head (ft)	Remarks
Corrugated Joint Opening, Silt at 85% Maximum Dry Density				
1	0.013	19.5	2.00	A small amount of infiltration occurred for a short
		5.0	4.47	period of time on application of the 6.54-ft head.
		7.5	6.54	A moderate amount of infiltration occurred for a
		1.0	9.99	short period of time on application of the 14.59-ft
		1.0	14.59	head. A partial washout occurred through both
		1.0	19.19	joints on application of the 19.19-ft head. Test
		0.5	23.79	was continued with no further infiltration
2	0.026	2.0	1.97	A small amount of infiltration occurred continuously
		3.5	4.24	during application of the 9.99- and 14.59-ft heads.
		4.0	6.54	No further infiltration occurred
		3.5	9.99	
		3.0	14.59	
		3.0	19.19	
		5.5	23.79	
3	0.054	4.0	1.98	A small amount of infiltration occurred for a short
		5.5	4.24	period of time during application of the 9.99-ft
		7.5	6.54	head. A partial washout occurred through both
		7.5	9.99	joints on application of the 14.59-ft head. The
		4.0	14.59	test was continued with small amounts of in-
		3.0	19.19	filtration occurring sporadically during each
		3.0	23.79	subsequent head
4	0.105	4.0	1.97	A partial washout occurred through the 6-in. joint
		97.5	4.26	8 hr after application of the 4.26-ft head. Test
		16.0	4.93	continued with only small amounts of infiltration
		8.0	6.54	occurring sporadically
		8.0	9.99	
		1.5	14.59	
		2.0	19.19	
		1.0	23.79	
5	0.105	19.0	3.01	A partial washout occurred through the 4-in. joint
		25.5	4.25	on application of the 8.84-ft head. Test continued
		5.0	6.54	with a total washout occurring through the 4-in.
		4.5	8.84	corrugation on application of the 19.65-ft head
		2.5	12.29	
		–	19.65	
6	0.130	1.8	4.01	A total washout occurred through both joints 7 min
		4.0	5.39	after application of the 14.59-ft head
		7.0	6.54	
		5.0	9.99	
		0.2	14.59	
7	0.158	19.0	1.99	A total washout occurred through both joints 9 min
		–	4.26	after application of the 4.26-ft head
8	0.158	1.5	1.99	A total washout occurred through both joints 2 hr
		2.0	3.02	after application of the 3.02-ft head
9	0.210	21.5	1.98	A total washout occurred through the 4-in. joint
		3.0	3.02	3 hr after application of the 3.02-ft head
10	0.260	1.0	1.08	A total washout occurred through both joints 8 min
		20.0	1.98	after application of the 4.26-ft head
		23.0	3.01	
		0.2	4.26	

TABLE 8 (Continued)

Specimen No.	Joint Opening (in.)	Test Duration (hr)	Test Head (ft)	Remarks
		Corrugated Joint Opening, Silt at 90% Maximum Dry Density		
11	0.158	23.2	3.01	A total washout occurred through both joints 3 hr
		24.0	4.26	after application of the 14.59-ft head
		10.0	5.39	
		5.7	6.54	
		8.0	7.69	
		7.7	9.99	
		7.7	12.29	
		3.0	14.59	
12	0.209	21.0	3.02	A total washout occurred through both joints 1 hr
		1.0	4.26	after application of the 4.26-ft head
13	0.209	21.0	1.97	A total washout occurred through both joints 2 hr
		5.0	3.01	after application of the 7.69-ft head
		15.7	4.26	
		3.5	5.39	
		3.5	6.54	
		2.0	7.69	
14	0.260	19.0	1.98	A total washout occurred through both joints 3.5 hr
		23.0	3.01	after application of the 19.19-ft head
		6.5	4.26	
		17.0	5.39	
		3.7	6.54	
		3.1	7.69	
		4.0	8.84	
		3.5	9.99	
		1.7	11.14	
		5.7	12.29	
		4.5	14.59	
		3.0	16.89	
		3.5	19.19	
15	0.260	15.7	1.97	A total washout occurred through both joints 1.0 hr
		71.5	3.01	after application of the 16.89-ft head
		23.3	4.27	
		6.7	5.39	
		7.5	6.54	
		8.0	7.69	
		7.5	9.99	
		8.0	12.29	
		7.7	14.59	
		1.0	16.89	
		Sample Slot Opening, Silt at 85% Maximum Dry Density		
16	0.050	5.5	5.39	A small amount of infiltration occurred continuously
		4.0	6.54	during application of the 12.29-ft head. A total
		19.0	7.69	washout occurred 6 hr after application of the
		4.0	9.99	14.59-ft head
		23.4	12.29	
		19.0	14.59	
17	0.053	18.5	4.24	A small amount of infiltration occurred continuously
		4.0	5.39	during application of the 7.69- and 8.84-ft heads.
		17.5	6.54	A total washout occurred during raising of the
		24.0	7.69	head, at 9.76 ft
		4.0	8.84	
		–	9.76	
18	0.095	18.5	5.39	A total washout occurred 4 hr after application of
		4.0	7.69	the 8.84-ft head
		4.0	8.84	
19	0.124	21.0	1.98	A small amount of infiltration occurred for a period
		23.5	3.01	of 40 min on application of the 7.69-ft head. A
		23.5	4.26	total washout occurred 5.2 hr after application of
		23.5	5.39	the 9.99-ft head
		8.0	6.54	
		24.0	7.69	
		5.2	9.99	
20	0.125	1.0	5.39	A total washout occurred 1 hr after application of the head (5.39 ft)
21	0.188	22.0	1.97	A total washout occurred 6.3 hr after application of
		23.5	3.01	the 5.39-ft head
		23.5	4.25	
		6.3	5.39	
22	0.250	1.6	1.97	A moderate amount of infiltration started 5 min after application of the head of 1.97 ft and continued for 1.6 hr when a total washout occurred

TABLE 9
SUMMARY OF INFILTRATION TEST DATA
(Corrugated Joint Opening, Lean Clay at 85 Percent Maximum Dry Density)

Specimen No.	Joint Opening (in.)	Test Duration (hr)	Test Head (ft)	Remarks
1	0.210	21.3	3.01	A total washout occurred through both joints imme-
		4.0	4.26	diately on application of the 19.19-ft head. No
		3.0	6.54	infiltration had occurred prior to the washout
		4.0	7.69	
		3.5	9.99	
		2.0	14.59	
			19.19	
2	0.210	5.4	5.39	No infiltration or washout occurred
		4.0	7.69	
		3.5	9.99	
		4.0	12.29	
		3.5	14.59	
		4.0	16.89	
		3.5	19.19	
		4.0	21.49	
		4.0	23.74	
3	0.260	7.0	5.39	No infiltration or washout occurred
		4.0	6.54	
		3.0	7.69	
		4.0	8.84	
		4.0	9.99	
		4.0	12.29	
		3.0	14.59	
		4.0	16.89	
		3.5	19.19	
		4.0	21.49	
		4.0	23.79	

Only 3 tests were performed using lean clay specimens, and they were performed with corrugated joint openings of 0.210 and 0.260 in.

DISCUSSION OF TEST RESULTS

Tests on M-F Sand and Uniform Sand

Figure 8 shows data from Tables 3, 6, and 7 on joint openings, heads, and occurrence of infiltration sufficient to cause total or partial washouts of the sand specimens. Figures 9 and 10 show a total washout in the M-F sand and uniform sand for the corrugated joint.

Although both partial and total washouts occurred in the corrugated joint tests using the M-F sand, no partial washouts occurred in the simple slot tests. The tortuous flow through the corrugated joints allowed a filter to develop within the joint which prevented the partial washout condition from developing into a total washout condition. This was verified by observing material in the valleys of the corrugations upon disassembly of the model after the test. In the simple slot test, this was not possible, and pronounced infiltration continued past the partial washout stage into a total washout condition.

No washout occurred in simple slot tests (9, l0, 11) with M-F sand and slots from 0.095 to 0.118 in. wide even under the highest test heads (23.79 ft), but partial washouts occurred under much lower heads (0.198 to 6.54 ft) in the tests with corrugated joint openings of 0.105 to 0.126 in. (WES tests 2 and 3 and ORDL test 4).

Test data in Figure 8 for the M-F sand (D_{85} = 0.035 in.)* compacted to 85 percent maximum density indicate that total washouts generally occurred in tests with either slots or corrugated openings of 0.125 in. or larger. (The only exceptions were WES test 3 with an 0.126-in. opening in which a partial washout occurred under a 1.98-ft head, and ORDL test 2 with an 0.158-in. opening in which a partial washout occurred when the test head was only 1.01 ft.) For uniform sand (D_{85} = 0.017 in.) compacted to 90 percent maximum density, Figure 8 shows that total washout occurred in 4 of 5 tests with corrugated openings 0.080 in.; only one partial washout occurred (in this test, the head was subsequently carried to 23.79 ft without further incident).

*D_{85} is the diameter of the maximum particle size for 85 percent of the sample.

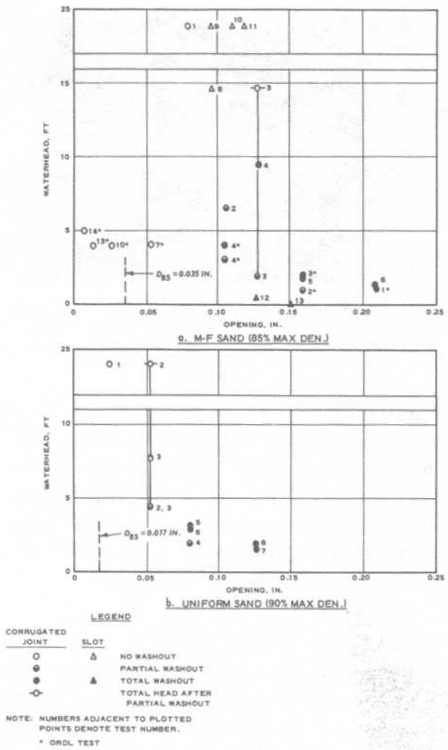

Figure 8. Infiltration through corrugated joint and slot openings of various sizes, sands.

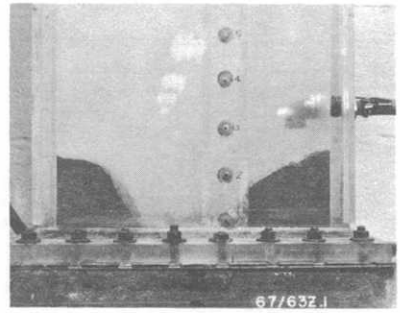

Figure 9. Total washout, ORDL test, M-F sand.

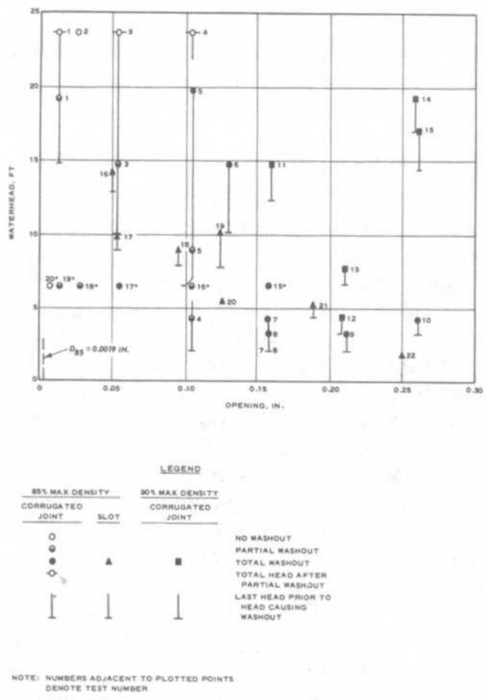

Figure 11. Infiltration through corrugated joint and slot openings of various sizes, silt.

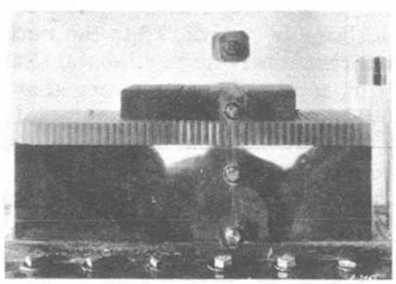

Figure 10. Total washout, WES test, uniform sand (counterweights inadvertently left in place in this one test, same test as shown in Figure 6).

The critical ratios of the D_{85} of the sands to the joint openings with respect to infiltration were:

$$\frac{0.035}{0.125} = 0.28 \text{ for the M-F sand}$$

$$\frac{0.017}{0.080} = 0.21 \text{ for the uniform fine sand}$$

These ratios are considerably smaller than the 1.2 ratio previously established in WES filter studies with slotted wood well screens (4). The principal reason for the

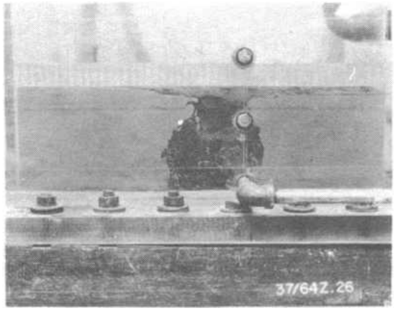

Figure 12. Total washout, ORDL test, silt.

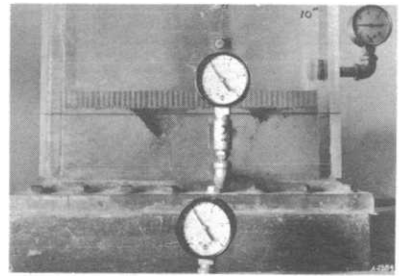

Figure 13. Partial washout, WES test, silt.

more restrictive criterion for the slotted well pipe is considered to be that, in the WES filter studies, the wells were surged; the surging consisted of rapidly changing the flow through the foundation sands, filter gravel, and well by changing the pressure from zero head to heads up to 10 ft a number of times in succession. Other contributing factors may have been that the filter materials were not compacted in the well screen tests as in the soil-infiltration tests, and that there were differences in particle shapes and sizes of the materials in the two investigations.

Tests on Silt

Figure 11 shows data presented in Table 8 on joint openings, heads, and occurrence of infiltration sufficient to cause total or partial washout of the silt specimens. Figures 12 and 13 show a total washout and a partial washout in the silt for the corrugated joint. These tests indicated the same results as those of sands in that no partial washouts occurred in the tests using slots, only total washouts, while both partial and total washouts occurred in the tests simulating corrugated joints.

In the corrugated joint and simple slot tests where the silt specimens were compacted to 85 percent CE-55 maximum dry density, partial washouts occurred in tests with corrugated joint openings as small as 0.013 in.; it was found in the WES tests that for joint openings of 0.053 in. and less, heads could later be increased to the maximum test head (23.79 ft) with only slight Infiltration occurring after the partial washouts. As joint openings were increased to 0.105 in. and larger, total washouts occurred at progressively lower heads, with two exceptions: (a) WES test 4 at a joint opening of 0.105 in. in which the head was increased to 23.79 ft following a partial washout at a head of 4.3 ft without further significant infiltration; and (b) ORDL test 16 with a joint opening of 0.105 in. in which a partial washout occurred on application of the head of 6.5 ft, but without further significant infiltration. On the basis of the corrugated joint tests, it appears that total washouts in silt would not occur if the joint openings were less than about 0.075 in. All 7 tests with the simple slot resulted in total washouts under progressively lower heads as the slot opening was increased. Total washouts occurred in 2 tests with 0.05-in. slot openings (the smallest openings tested), under heads of 9.8 and 14.6 ft (tests 16 and 17).

Five tests with silt specimens compacted to 90 percent maximum density in which corrugated joint openings ranged from 0.158 to 0.260 gave rather erratic results with respect to the heads producing total washouts, but the general trend indicated that the soil specimens with greater density could withstand much higher heads before total washout occurred.

Tests on Lean Clay

Only 3 tests were performed with lean clay specimens compacted to 85 percent CE-55 maximum dry density (Table 9). In 2 of the tests the corrugated joint opening was 0.210 in., and in the third test the corrugated joint opening was 0.260 in. In the test with the 0.260-in. opening, the head was raised to 23.79 ft without any infiltration occurring. This was true also of one of the tests with an 0.210-in. opening; but in the other test, a total washout occurred when a head of 19.19 it was reached. The reason for this washout is not known. The last two increments of head applied were about double those used in the other 2 tests, and this may have had some effect in producing the washout. These few tests on a lean clay of relatively low plasticity when compared to the performance of the silt clearly demonstrate the effect of a comparatively small increase in plasticity in lessening the probability of soil infiltration through pipe joints.

CONCLUSIONS

The soil infiltration tests reported in this investigation demonstrated that a number of factors influence the degree of soil infiltration through pipe-joint openings. The most significant factors appeared to be (in addition to the obvious factor of opening size): grain size, plasticity, and density of the surrounding soil, and the nature of the flow path through the joint opening (i.e., whether tortuous and long as through a corrugated joint, or direct and short as through a simple slot). The effects of vibration were indicated to be significant, but no quantitative data of use to field application were developed from the few incidental and unsophisticated experiments in which various types of vibration were applied to the specimen container.

Measurements of openings between coupling bands and corrugated pipe showed that at some locations beneath the band the openings were from 0.18 to 0.35 in., depending on the size of the pipes. The infiltration tests showed that total washouts would occur with joint openings larger than about 0.125 in. for the M-F sand and larger than about 0.080 in. for the uniform sand. For stability, this would mean the ratio of D_{85} to joint openings should be not less than 0.28 for the M-F sand and 0.21 for the uniform sand. It is thus indicated that severe infiltration of these materials could occur in portions of the corrugated banded joints where such materials were compacted only to 85 or 90 percent CE-55 maximum dry density.

Although total washouts generally occurred in the silts compacted to 85 percent CE-55 maximum dry density where openings were 0.105 in. or larger, two exceptions at smaller openings indicated that the critical opening was about 0.075 in. The tests with the silts at two different densities demonstrated the benefit of greater compaction in preventing excessive soil infiltration. It is logical that the same benefit of increased density would be gained with the sands which, of course, would be enhanced should the sand particles be angular rather than rounded.

The few tests with the lean clay when compared with those of the silt showed that even a relatively small increase in cohesion (plasticity) in a fine-grained soil would prevent soil infiltration. It is concluded that fine-grained soils with a plasticity index greater than about 12 would not be susceptible to infiltration when compacted at optimum water content to 85 percent maximum CE-55 density.

RECOMMENDATIONS FOR FUTURE RESEARCH

Further studies are needed on the sizes and extent of joint openings in field installations, and additional laboratory tests are needed to establish grain-size criteria for cohesionless backfill. Consideration should be given in any future testing programs to developing a simpler test apparatus with flow through the test specimens exiting through a single joint opening in the base. The joint opening might have either a corrugated or a straight configuration with a length of about 4 in. in the direction of flow.

REFERENCES

1. Final Report on Study of Watertight Drainage Pipe Joints. U. S. Army Engineer District, Savannah, Nov. 1955.
2. Brown, D. N. Report of Trip to Savannah District and Hunter, Pinecastle, and MacDill Air Force Bases in Connection with Drainpipe Study. U. S. Army Engineer Waterways Experiment Station, Miscellaneous Paper No. 4-144, Vicksburg, Miss., March 1955.

3. Guide Specification for Military Construction; Storm-Drainage System. U. S. Army, Office of the Chief of Engineers, CE-805.01, Washington; first published Dec. 1958, current revision published Sept. 1967.
4. Investigation of Wooden Well Screens for Grenada, Enid, and Sardis Dams. U. S. Army Engineer Waterways Experiment Station, Tech. Memo. No. 3-250, Vicksburg, Miss., Feb. 1948.
5. Nettles, E. H. WES Investigational Project, Soil Infiltration Studies of Corrugated Steel Pipe. U. S. Army Engineer Waterways Experiment Station, Miscellaneous Paper No. 4-657, Vicksburg, Miss., July 1964.
6. Soil Infiltration Studies of Corrugated Steel Pipe. U. S. Army Engineer Ohio River Division Laboratories, Interim Rept., Cincinnati, Ohio, Jan. 1964.

Structural Failure of Western Highways Caused by Piping

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• PIPING, a type of subsurface erosion, is an extremely destructive erosive process which attacks certain structures of man and landforms of nature (1, 2). The objective of this report is to detail the current and potential damage of piping to the highways in the West as well as to point out some ways in which this damage may be minimized. A further report is in preparation presenting physicochemical-mineralogical analyses and the detailed mechanisms of piping and of subsidence in the western drylands.

Much has been written on the details of erosion, including the formation of gullies and arroyos, yet the geologic literature contains only sparse note of piping. Local attention has been given to this phenomenon by soil scientists (3-7), but no comprehensive treatment of the subject is currently available. Recently, however, Australian and New Zealand scientists became concerned about the frequent failure of small earth dams. It was determined that 8 percent of the dams constructed failed as a result of piping (8). The studies led to an extensive examination of the piping process as a major cause of dam failure and resulted in a Colloquium on Failure of Small Earth Dams in Melbourne, Australia, November 16-19, 1964.

Piping occurs with the greatest frequency in the world's drylands (arid and semiarid regions) and, in these regions, it occurs most commonly in valley alluvium which has been or is being trenched by gullies. Piping also occurs in the loess areas of the Mississippi Valley, where rainfall may exceed 50 in. per year (9), in New Zealand, where rainfall is about 28 in. per year (6), and in Africa, where the rainfall is about 30 in. per year (10). In the United States, piping has been observed at numerous sites. However, this report deals principally with piping in the western drylands, particularly in Colorado, New Mexico, Arizona, Utah, and Nevada.

Piping is basically the development of subsurface drainage systems in earth materials to a depth no greater than the nearby base level of drainage. It results in a surface expression much like miniature solution depressions in limestone or dolomite terrains. Three different types of piping have been recognized, depending on their mode of origin: (a) desiccation-stress crack, (b) entrainment, and (c) variable permeability subsidence.

An associated diagenetic phenomenon which causes fracturing of alluvium is that of subsidence or "collapsing structure"; the latter term being used by South African investigators (11-13). In essence, this subsidence results from the breakdown of secondary aggregates, which occurs as a result of saturating previously unwetted low-density earth materials. In the drylands, alluvium, colluvium, and loess are particularly susceptible to subsidence when wetted. Subsidence of this type may result in a volume decrease of as much as 13 percent (14) and a lowering of the ground surface by 10 to 15 ft (15), forming sinks (14) or voids beneath the land surface which provide routes for the movement of subsurface water and the development of pipes. Thus, subsidence may be a causative factor in the development of either the desiccation-stress crack or the variable permeability-subsidence types of piping.

The dominant type of piping currently damaging highway structures in the western drylands originates from desiccation cracks. Runoff enters a drainable desiccation crack at the land surface. As the runoff moves downward it erodes and enlarges the

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crack walls. Where the hydraulic gradient is sufficient, fine-grained sediment is transported in suspension along the crack to appear at an incipient pipe outlet in a gully wall, an arroyo side slope, or embankment. This type of pipe may also occur as a result of localized subsidence, due to saturation of surficial sediments, forming sinks or stress cracks; or, rarely, an animal burrow or rotted-root tube. These pipes have visible inlets and outlets except in those instances where the rainfall saturates the ground thereby causing slumping and temporary closure of pipe ends.

The entrainment type of piping occurs chiefly during the dewatering of building foundations, or during the rise of impounded water behind levees and dams (16). These pipes and their associated boils are induced where newly created large hydraulic heads cause channelized subsurface flow, with entrainment of water-saturated earth materials, to a discharge point in an excavation or to the down-gradient side of a dam or levee. The entrainment form of piping rarely produces open subsurface pipes; however, it may transport enough earth material to cause the collapse of the overlying surface with sudden destruction of the superjacent structure (1, 17).

The variable permeability-subsidence type of piping, which produces open pipes, results where a sufficient hydraulic head exists to move water through a stratum with sufficient velocity to transport dispersed clay or even silt and sand at the face of a gully, side of an embankment, or a steep side slope. Visible outlets develop, for example, at a gully wall and grow headward away from the wall.

Where piping is extensively developed, a characteristic and easily recognized topography termed pseudokarst (1) is generally present (Figs. 1, 2). The landform appearance is that of a miniature limestone or dolomite terrain marked by solution features such as sinkholes, natural bridges, caves, blind valleys, and haystack-shaped hills. The term "karst" is applied to this landscape; hence, for the piped landscape the term "pseudokarst" is applied. Although both the desiccation-stress crack and the variable permeability-consolidation types of piping simulate solution erosion in limestone or dolomite, at present there is no evidence that solution plays any important role in the piping process in sedimentary materials.

Figure 1. Aztec Wash showing pseudokarst developed in trenched and piped alluvium. Two culverts which discharge near north base of roadway fill are visible. Concentration of drainage runoff at edge of roadbed accelerates piping and gullying and contributes directly to undermining of highway. Tableland in background would provide excellent roadway in lieu of present location (see Figs. 13, 18, and 19).

PIPING PROCESSES

No single factor determines whether the desiccation-crack type of piping will occur at a given site in the western drylands; rather, it is the interaction of several physicochemical-mineralogical factors which determines whether a given sedimentary material will or will not pipe. However, four minimum requirements exist: (a) enough water must be available to fill drainable cracks; (b) the strata must be montmorillonitic; (c) the strata must desiccate thoroughly, if only seasonally; and (d) there must be an outlet for drainage. Furthermore, the higher the sodium to calcium-plus-magnesium ratio, the less the stability of crumb aggregation, the less the vegetative cover, and the less the slope, then the greater is the probability of desiccation-stress crack piping, and the greater its intensity where it does occur. Where uneven and differential subsidence occurs, the only minimum requirement may be that of a sufficient hydraulic gradient.

Although a sufficient hydraulic gradient must exist, a gradient which is adequate to facilitate piping in one stratum may not be adequate for another. Gullies, in alluvium, only 1½ ft deep have been observed with pipe outlets in their walls. Whether pipes outlet somewhere in the gully wall or at the base of the gully wall appears to depend upon the depth to which the desiccation-stress cracks occur, or the presence of permeable layers interstratified with less permeable layers above the gully floor.

Montmorillonite, mixed-layer illite-montmorillonite, and illite are the clay minerals which are nearly ubiquitous in the piping earth materials examined to date (1, 2). Recently obtained cation exchange capacity data indicate that the montmorillonite content of piping materials ranges from 20 to 50 percent on a dry-weight basis. Pure montmorillonite is capable of volume changes of up to 1600 percent in going from the air-dry state to maximum moisture sorption (18). Thus, surficial sediments containing 20 to 50 percent montmorillonite can be expected to crack widely and deeply upon thorough desiccation, such as occurs in the western drylands. In humid areas, similar materials would: (a) seldom have the opportunity for thorough desiccation, (b) have been wetted to depth so that further subsidence as a result of saturation would not be

Figure 2. Pseudokarst developed in dense, consolidated shaly bedrock of triassic Chinle formation; in this area it is largely derived from volcanic ash falls, weathered to red, green, and white bentonific shales. Undrained, plugged sinkholes are partially filled with water from recent rainstorm.

expected to occur, (c) not be largely sodium saturated unless adjacent to salt water, and (d) commonly be kaolinitic (or illitic) rather than montmorillonitic. Thus, in humid climates similar materials would not be expected to pipe. However, in sub-humid regions which have extended, hot, dry summers, weather conditions may cause cracking to occur. It is believed that desiccation caused by summer droughts largely explains the incidence of piping in areas which have 20 to 50 in. per year average rainfall.

Soluble and exchangeable sodium affect the susceptibility of a stratum to piping in a number of ways. The alluvial mineral grains are principally stabilized into crumbs by clay coatings and by the cementing action of iron and manganese oxides, silica, and calcite. These crumbs, which form the bulk of alluvial materials susceptible to piping, are stable under existing dryland weather and overburden conditions. As long as they remain dry, or at least unsaturated, they bear heavy loads but rapidly lose this ability upon saturation. The higher the sodium to calcium-plus-magnesium ratio, the greater will be the montmorillonite volume change during swelling. The greater the amount of swelling, the more thoroughly the cements and clay coating will be fractured. The greater the sodium to calcium-plus-magnesium ratio, at a fixed total soluble salt content, the more readily the montmorillonite will disperse and flush out in percolating waters. This removal of clay in percolating water may be referred to as colloidal erosion.

Dispersed montmorillonite clay particles may wash away in suspension, even through thin cracks in consolidated shale bedrock, or between larger sand-sized particles in unconsolidated sedimentary beds. Where fractures are enlarged sufficiently for turbulent flow to occur, erosion becomes greater through corrasion and the rate of localized erosion is thus greatly increased. A by-product of the clay dispersion, as a result of high soluble sodium content, is a decreased permeability of the ground surface which increases the amount of runoff available to enlarge pipes. Vegetative growth is also decreased by high soluble sodium contents.

Vegetative denudation, whether by overgrazing, recurrent burning, or climatic change, promotes piping (19, 20). This is a result of a decrease in surface permeability due to: (a) decreasing contents of organic matter; (b) increased breakdown of crumb structure at soil surface due to raindrop impact on unprotected surface; (c) the raising of surface and subsurface temperature which, in turn, results in greater desiccation; and (d) greater runoff due to loss of the obstruction provided by vegetative cover.

Some additional observations may be noted concerning the significance of desiccation-stress cracks to piping. Extensive piping occurs in alluvium derived from the cretaceous Pierre shale along US 85-87 south of Colorado Springs, Colo. However, piping has not been observed in irrigated fields in this area. More significantly, piping does not even occur in the highway right-of-way adjacent to the irrigated fields due, we believe, to subsurface movement of moisture from the irrigated fields. Town Dump Wash, near Bayfield, Colo., meanders on alluvial fill between two low hills of paleocene Animas shale. The piping intensity in the alluvium is distinctly less, alternatively on the one and then the other side of the Wash, depending on the nearness of the Wash to the Animas bedrock. It is presumed that subsurface moisture moves from the Animas shale into the alluvium and that the amount of moisture is adequate to support a good grass cover only in a reasonably narrow strip of alluvium adjacent to the shale. The density of the vegetative cover tends to be inversely related to piping intensity.

In a piped area, the frequency of pipe inlets commonly increases toward the arroyo banks. This results from an increased contribution of the gravity stress factor to the desiccation-stress cracks which parallel an arroyo bank. When the ground is wetted the major swelling movement is toward the unconfined or free face of the arroyo. Less swelling occurs with depth than near the surface due to moisture deficiency at depth. When desiccation occurs again, the cohesive force of the sediment is not sufficient to pull the arroyo bank back to the vertical so cracks develop which are largely parallel to the arroyo bank.

Where subsidence, due to saturation of the ground, occurs at depth in low-density earth materials such as alluvium or loess, underground cavities may result as reported by Turnbull (21). Stress cracks may subsequently open up to the surface and serve as conduits for water to pass from the surface down to the subsided area. Given opportunity for lateral ground water flow to a nearby ditch, canal, or arroyo, pipes may then develop at depth in the formation (22).

Piping has been observed in the grass-covered alluvium of portions of Town Dump Wash and in Montana (23) and Uganda (10). In the Town Dump Wash alluvium, a large pipe outlet was found at a depth of about 8 ft in a layer of alluvium slightly more sandy and permeable than the material between it and the land surface or the exposed strata below it. Thus, the layer of higher permeability serves as a drain during snow-melt runoff or rare prolonged rains. The horizontal transmissibility appears adequate to develop a sufficient water velocity to carry dispersed clay out to the gully. Locally, the hydraulic gradient may be adequate to move silt and sand as well at the headward end of the pipe. Bishop (10) reports that ". . . during heavy rainfall [following a protracted dry spell] the water-table may rise rapidly and when supply exceeds drainage by a sufficient amount 'perched water-tables' build up above layers of relatively impermeable strata. If the rainfall is sufficiently concentrated the surface of the water-table develops a slope towards lines of [surface] drainage. Such a build up of ground-water results in rapid subterranean flow into gully heads, undercutting both unconsolidated beds and any surface cover of vegetation. The gullying [piping] proceeds in steps above each relatively impermeable band and temporary tunnels up to 3 feet in diameter can be seen leading off from the heads of active gullies. With the collapse of tunnels [pipes] gullies may advance headwards as much as 10 or 15 feet in a single storm."

The development of pipes away from a gully wall, without visible inlets, may also occur where a gully or drainage ditch intercepts a water table (Fig. 3). Several such instances have been noted in the irrigated areas of Wyoming and Nebraska. As the pipes grow in length and diameter, settlement of the overlying material may take place, particularly after an extended rain or snow melt which may wet the alluvium well below the surface. The water adds to the overburden weight and decreases the crumb strength as previously noted. Subsequently, surface runoff collects in the subsidence depressions causing either further subsidence or development of vertical pipes down to the more or less horizontal subsurface pipes by: (a) colloidal erosion, i.e., carrying out dispersed fines; (b) enlarging a desiccation crack; (c) enlarging a stress crack (resulting from subsidence); or, rarely (d) enlarging an animal burrow or rotted-out root

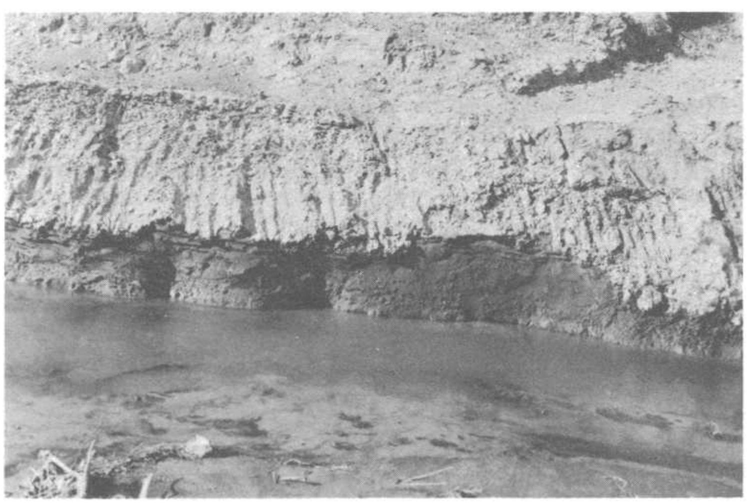

Figure 3. Pipe outlets near base of gully wall. Subsidence caused by irrigation return underflow maybe responsible for development of these pipes in Oligocene Brule formation White River group.

Figure 4. Drainage ditch showing result of piping and subsidence. Ditch resulted from heavy rainstorm. Materials affected are greenish-gray silty alluvium derived from cretaceous Mancos shale.

tube. In places it appears that whole gullies have formed from the collapse of pipes of the variable permeability-subsidence type. In some situations, it appears probable that both subsidence and desiccation-stress cracks combine to facilitate piping. The drainage ditch shown in Figure 4 may have formed in this manner.

The confusion in the literature between "collapse structure," a form of subsidence, and piping is exemplified by studies of the White Silts loess area near Kamloops, B. C., Canada. Reports by Cockfield and Buckham (5), and Buckham and Cockfield (24) describe this area as a pseudokarst (although they do not call it that) produced by piping. However, a later report on this same area by Hardy (14), identifies collapse structure (subsidence) as the phenomenon responsible for the distinctive topography of this area. Without having seen the area it seems likely to us that both processes are responsible for the caved in and piped area at Kamloops. In reality, subsidence greatly increases the intensity of piping in any earth material subject to piping by: (a) concentrating runoff, (b) developing stress cracks, and (c) developing partial lateral pipes by uneven subsidence and differential consolidation between different strata.

Observations of piping phenomena and our interpretation of the processes involved allow piping to be categorized into three types: desiccation-stress cracks, variable permeability-subsidence, and entrainment. The former is the dominant type in the western drylands, although the variable permeability-subsidence type has also been observed widely; the latter type occurs only in the case of water storage or dewatered structures.

DESTRUCTIVE EFFECTS OF PIPING ON HIGHWAY STRUCTURES

The authors have not had the opportunity of making a careful and exhaustive determination of the locations of all the sites where highways, bridges, culverts, and other highway structures are imperiled by piping. However, in the course of field work related to the development of a better understanding of drylands erosion, a number of such piping sites have been observed. The locations of many of the observed sities are shown in Figure 5, and some are pictured in the photographs which follow. However, the list is far from complete. Many more sites imperiled by piping could doubtlessly be found by a systematic search along the highways crossing the geologic formations known to be susceptible to piping, or by careful study of large-scale areal photographs of these susceptible areas followed by field checking. Examples that follow are chosen to show how piping affects abutments and wingwalls, piers, drainage ditches, culverts, embankments, and roadways.

So far as we have observed, bridge abutments and wingwalls are the structures most often imperiled by piping. Deck drainage is generally permitted to drain through short vertical metal pipes and spill directly onto the abutment slope or to run down the abutment wall through joints between the bridge deck and road surface. These

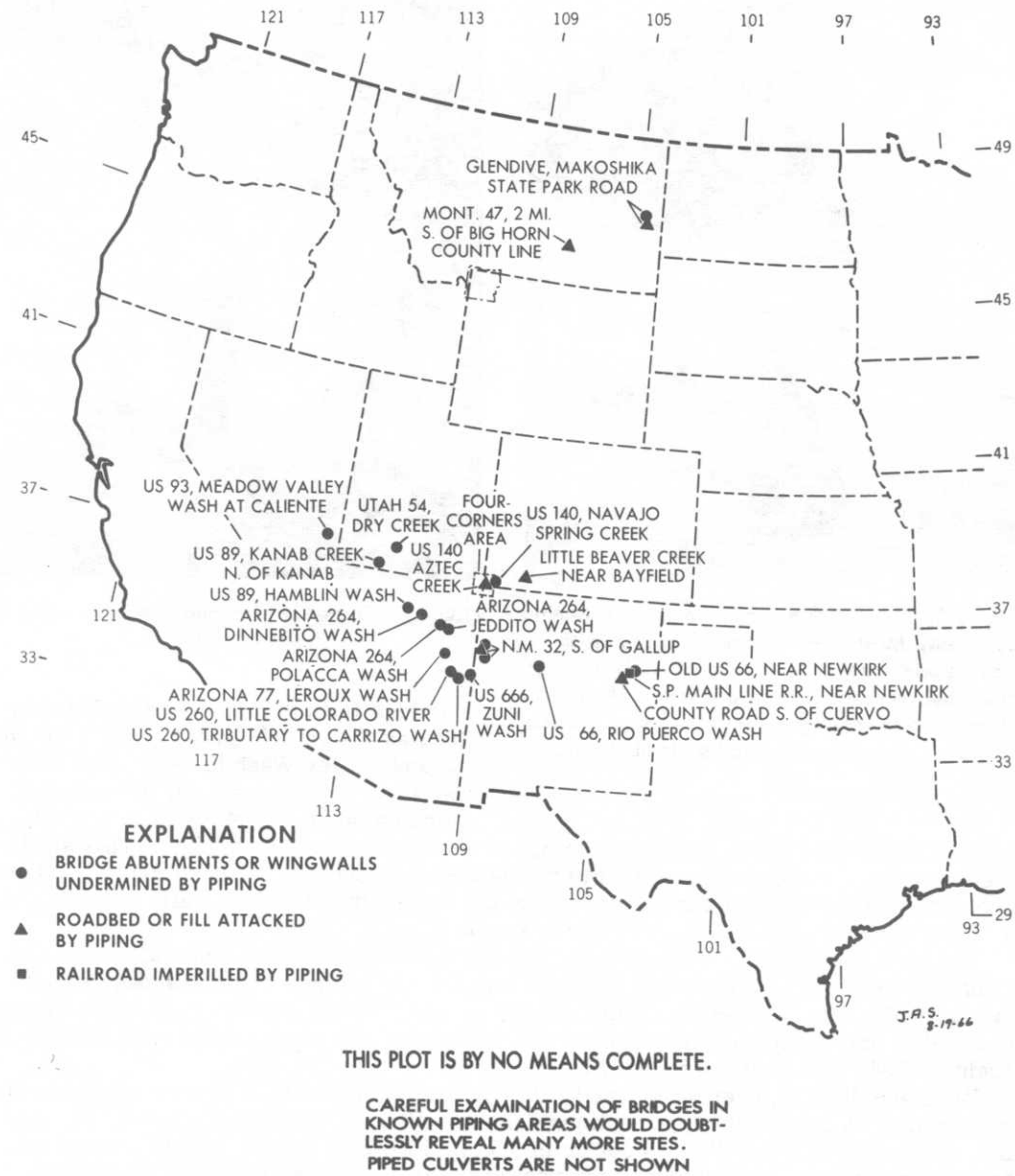

Figure 5. Location of bridges, roads, and railroad embankments or rights-of-way known to be imperiled by piping.

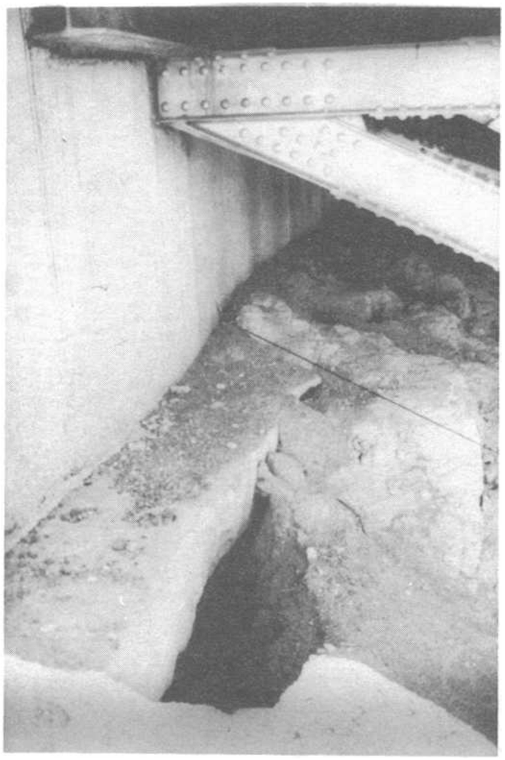

Figure 6. Pipe intakes beginning to undermine abutment. Most water that gains access to abutment slope runs down abutment wall through crack between bridge deck and road surface. As pipes become enlarged, roofs collapse leaving gaping holes beneath and beside abutment.

Figure 7. Outlet of pipe, one of whose intakes is shown in Figure 6.

practices initiate piping in the abutment slope as shown in Figures 6 and 7 for the Meadow Valley Wash bridge. The initial result, as well as the first indication of piping on an abutment slope, is the cracking and settlement of the road surface where it abuts against the bridge. This settlement is often erroneously assumed to be the result of secondary consolidation. On the contrary, the removal of earth from under and behind the abutment walls by piping is a common cause of the settlement of the roadbed.

Figure 8 shows the location of a pipe which is endangering the wingwall of US 666 Zuni Wash bridge. As this pipe enlarges and its roof collapses, stress-desiccation cracks will appear between the pipe and the bridge. These cracks can then be expected to develop into secondary pipes which will undermine the wingwall and abutment (Arizona Highway 264 Polacca Wash bridge, Fig. 9).

Drainage ditches, even when lined with concrete, commonly fail in areas where piping occurs. A seriously piped, unlined drainage ditch is shown in Figure 4. A similar fate of a concrete lined ditch is shown in Figure 10. This is only one of several such cases where piping has destroyed the usefulness of lined drainage ditch structures.

Culvert installations are destroyed with a frequency similar to that of drainage ditches. The complete loss of a section of road adjacent to a culvert near Newkirk, N. Mex., is shown in Figure 11. Note stress cracks over pipe in foreground and small side gully in left foreground formed by collapse of a branch pipe. In Figure 12, the destructive effects of piping plus the effect of overtopping of the road by runoff is illustrated. Any concentration of runoff for passage through a culvert greatly accelerates piping between the culvert outlet and the nearest gully. By collapse of the

Figure 8. Elongate pipe intake of piping system endangering abutment.

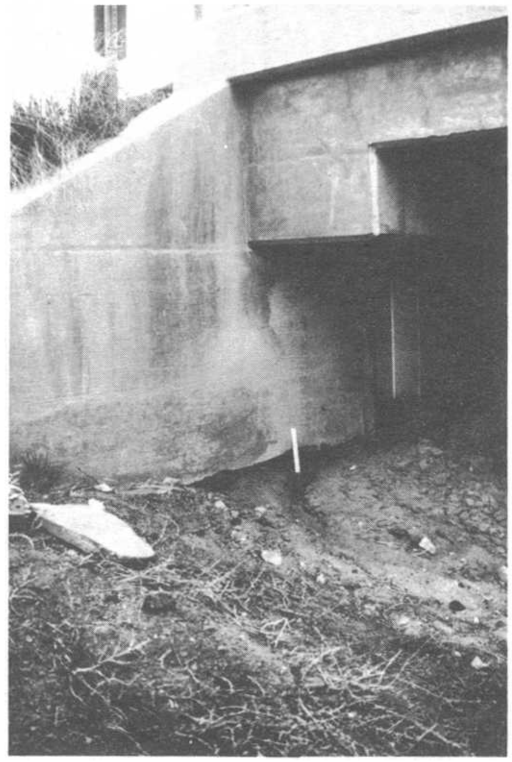

Figure 9. Abutment and wingwall showing outlet of pipe undermining wingwall. Larger and deeper pipes are found elsewhere under abutments of bridge.

overburden, a gully may advance towards a culvert from zero to 15 or 20 ft per rainstorm. The considerable depth of some of the gullies which have grown between culvert outlets and the gully system present at the time of road building is shown in Figure 13. Here, stress-desiccation cracks extend almost to the roadbed itself and small pipes extend under the highway to discharge into an 18-ft gully. Already, cracks are present in the roadbed over this culvert as a result of subsidence and of settlement due to loss of support where the subgrade has been piped. Figure 14 shows the situation which precedes the development of a deep gully as shown in Figure 13. Note the pipe inlet directly under the culvert end in Figure 14, several feet from edge of nearest gully. Several pipe inlets appear between the culvert outlet and the short side gully on the roadward side of the arroyo. These sinkholes are interconnected by an extensive pipe system draining to the deep gully north of the highway. The major pipe outlets occur at the contact of the gully wall and the gully floor.

Surface roadbeds are most commonly damaged by increased piping caused by spilling the runoff collected by drainage structures, especially culverts and bridges, onto the land surface adjacent to these structures. Even greater difficulty is encountered in maintaining graded roads on materials subject to piping. This is illustrated in Figure 15 which shows a section of abandoned Navajo Highway 14. This stretch of road, which is one of the most intensively piped areas that we have seen, was constructed on fill derived from the triassic Chinle formation. Some strata of the Chinle formation pipe more intensively than any other bedrock we have observed. The intensity of piping of this roadbed is believed to be due to subsidence, which, as previously pointed out, greatly enhances piping intensity. Another example of an unsurfaced road in current

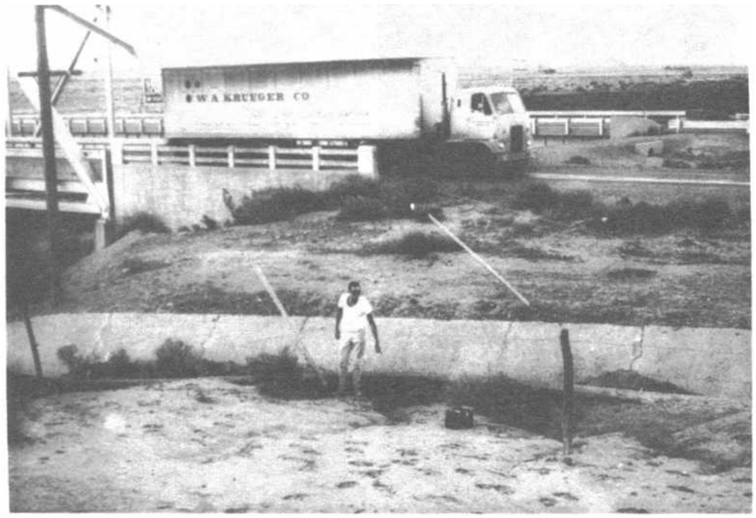

Figure 10. Concrete-lined highway drainage ditch, which failed due to piping. Concrete liner is now totally useless for intended purpose.

Figure 11. Culvert and drainage ditch are undermined by piping. Note collapse of ditch bed into underlying pipe. This is the beginning phase of a deep gully. Piped material is alluvium derived from triassic Chinle shale.

Figure 12. Runoff from intensive storm exceeded culvert capacity so that water overtopped road and piping developed around culvert. Culvert would have failed due to piping even if no overtopping of road had occurred as nearby road drainage ditches piped extensively.

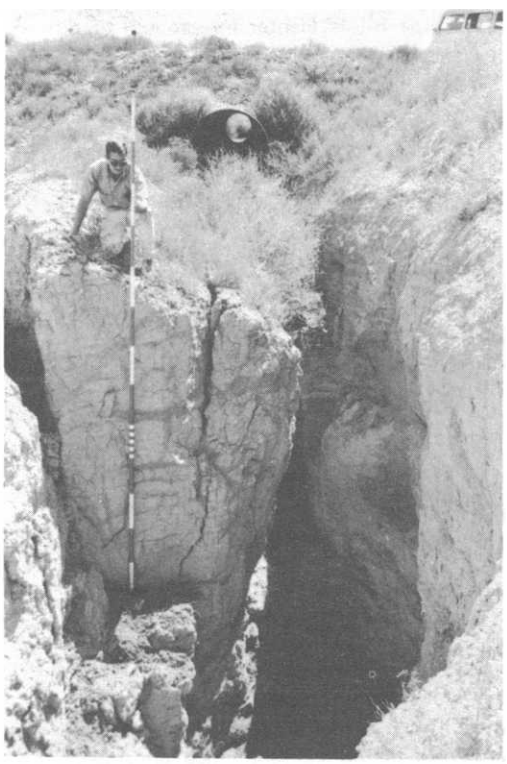

Figure 13. Culvert-concentrated drainage spill has created gully by piping and subsidence, which seriously threatens stability of roadway. (See Figs. 18 and 19, which graphically depict the conditions at this site.)

use, but that requires continuous maintenance because of piping, is shown in Figure 16.

An entire 6-mile stretch of US 140 is currently imperiled by piping. This section of road was built on Aztec Wash alluvium and is located southwest of Cortez, Colo., in the 4-Corners area (Fig. 5). This section of US 140 was only put into service in 1963, but already it is being undermined by pipes in several places (Figs. 13 and 14). Already, highway maintenance crews are making repairs to the new highway as it settles and cracks, especially on the arroyo side. The view in Figure 17 is fairly typical and shows settling and cracking of the roadbed as a result of piping and subsidence in the subjacent alluvium on the side of the road nearest to the arroyo. This figure also shows a large patch needed to restore the damaged road surface. In other places along this 6-mile stretch, several large asphalt repair patches have been emplaced one atop the other where settlement of the roadbed is even more rapid than at the site pictured.

The results of the desiccation-stress crack piping process as well as the destructive effect of piping on roadways are shown in Figures 18 and 19. The medial gully of Aztec Wash meanders from side to side in the alluvial fill of this bedrock

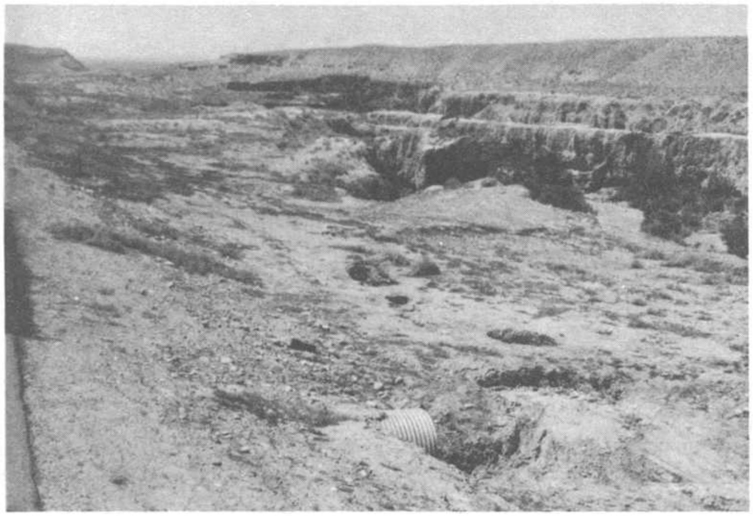

Figure 14. Sink, or pipe inlet, developed at toe of highway embankment as a result of spilling the collected drainage onto ground at culvert outlet. Note pipe inlets (center foreground) of culvert outlet. Note also edge of big asphalt patch in road pavement emplaced to repair damage caused by pipe subsidence.

Figure 15. A six-ft tall man is standing on ledge in pipe 10 ft from bottom of pipe inlet. Materials are red and green shale of the triassic Chinle formation. Both piping and subsidence are responsible for the erosion shown here.

Figure 16. Old US 66 is in center of photo and mainline track of Southern Pacific Railroad, which itself is undermined by piping in this area, is just beyond highway. Pipe intakes beside road in front of truck are 2 to 3 ft deep.

Figure 17. Cracks reflect settlement of roadway shoulder in response to piping and subsidence in subjacent alluvium on which highway was constructed. In the future, this roadway can be maintained only at greatly increasing costs. An alternative and cheaply maintained right-of-way may be found on stable bedrock upland on either north or south side of Wash.

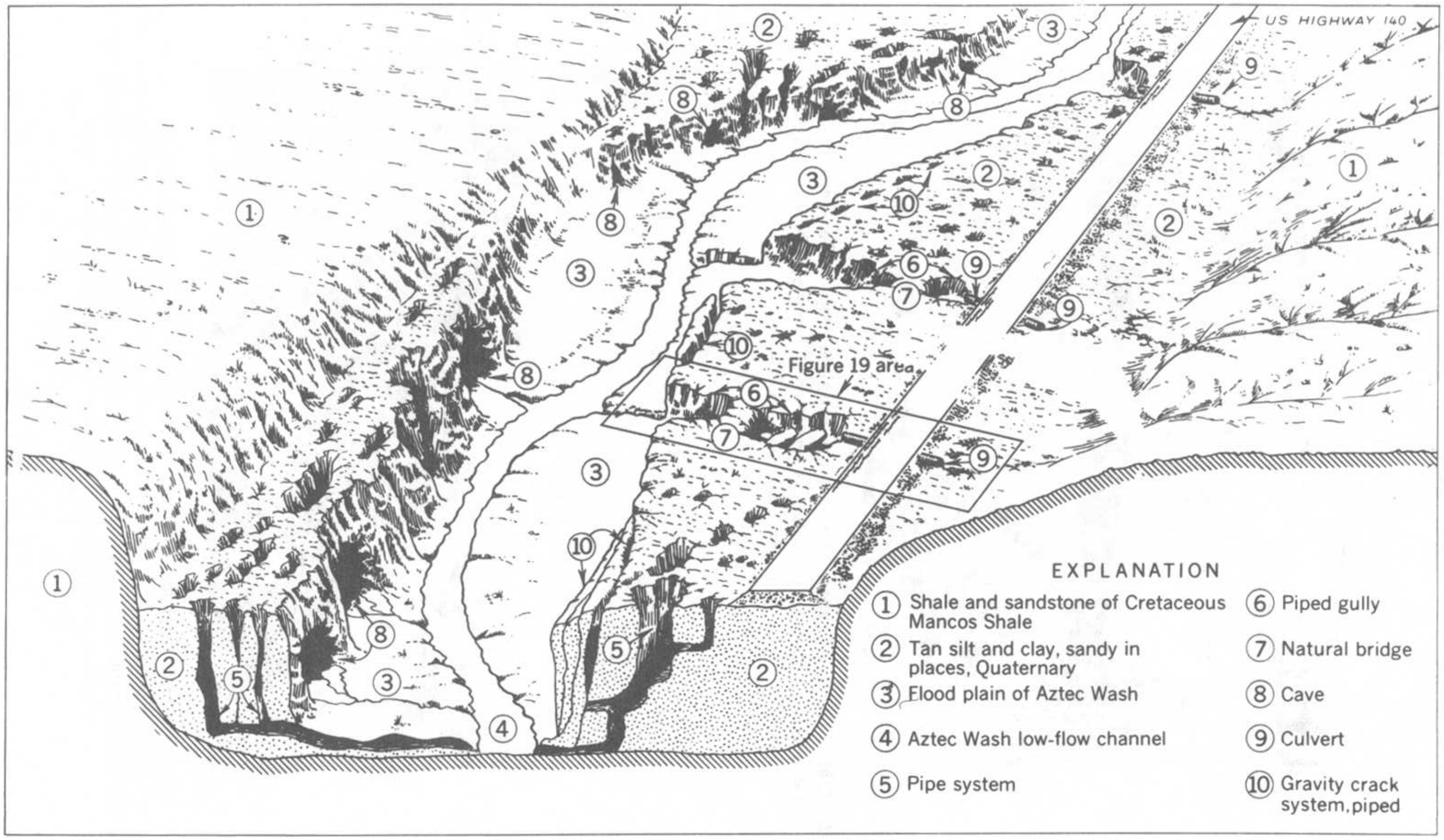

Figure 18. Idealized block diagram of Aztec Wash showing dissected and extensively piped valley fill, old bedrock surface and channel, highway, and drainage system. (See Figs. 1, 13, and 19.)

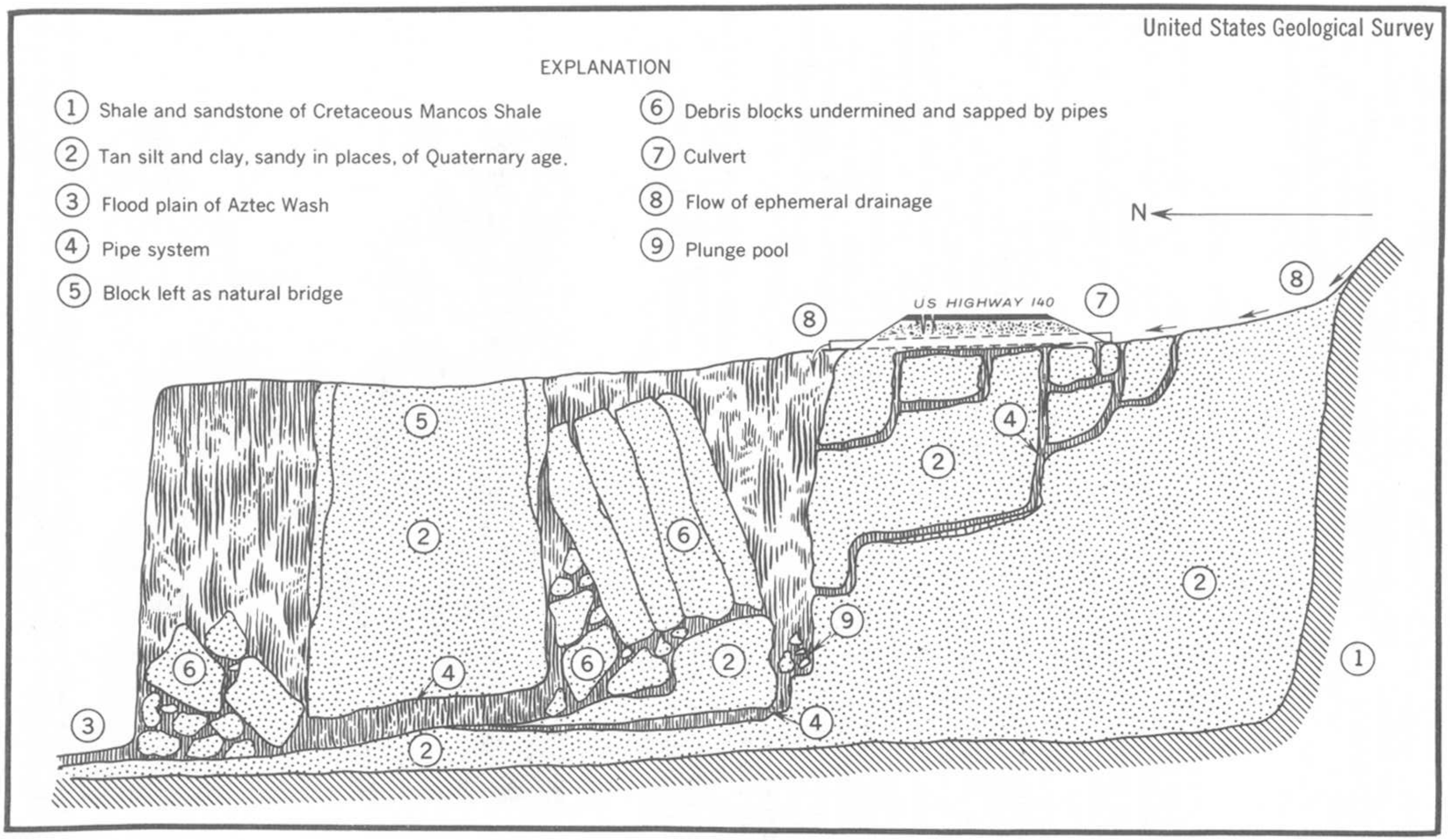

Figure 19. Idealized north-south cross section under US 140 built on Aztec Wash valley fill. Note incipient piping system beneath roadway. See Figure 18 for setting.

valley, and in places it has eroded the valley fill to expose the nonpiped bedrock. The valley erosion probably had its origin in the 1880's like other such major gullies of the Southwest. The current rate of erosion and removal of the valley fill is not accurately known but it is fairly rapid. Furthermore, the building of the highway and the concentration of surface runoff via culverts have accelerated erosion on the south side of the valley fill. Figure 19 shows a portion of the idealized block diagram of Aztec Wash in detail. In particular, the manner in which large blocks of alluvium are undercut is illustrated.

PREVENTATIVE AND REMEDIAL MEASURES

Although piping is most frequently observed in alluvium, it is by no means confined to this material. Piping also occurs on the crowns and slopes of miniature badlands formed from shales or, rarely, from other argillaceous rocks (Figs. 1 and 2). Thus, even the fact that a road is constructed on and from bedrock is no guarantee that it will not be subject to piping.

In order to minimize maintenance and relocation costs, as well as possible loss of life as a result of roadway failures attributable to piping, surveys of piping incidence should be made prior to road construction. The most desirable procedure is to select a route that will avoid construction on and with earth materials subject to piping. This could have been readily done in the care of the 6-mile stretch of US 140 which is being damaged and may be destroyed by piping. Figures 1 and 14 show that the tableland on both the north and south sides of the valley is relatively level. Since piping does not appear to attack this particular bedrock, initial choice of the tableland would have avoided both expensive maintenance and eventual relocation costs. Where suitable nonpiping bedrock is available, as in this case, relocation may be less expensive in the long run than maintenance of the present roadway. Where it is not practical to avoid building a road on materials susceptible to piping, several measures can be taken to minimize or prevent piping damage.

A difficult problem is that of leading culvert or roadway drainage to a nearby gully without increasing piping intensity in the process. One method would be to dig a sloping trench from the downslope side of the road to the nearest gully floor and to lay a closed conduit therein with a minimum slope to minimize the possibility of piping

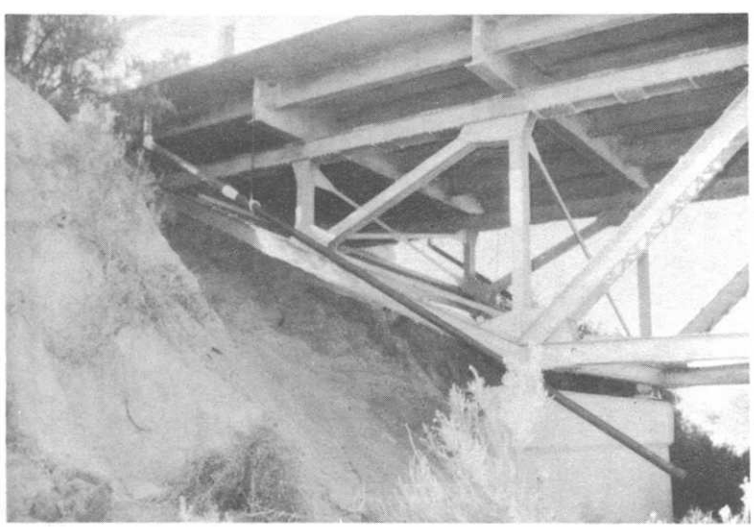

Figure 20. Metal pipe is carrying bridge-deck discharge directly into channel where it can do no harm instead of allowing it to drop onto abutment slope. Note companion discharge pipe on opposite side of bridge.

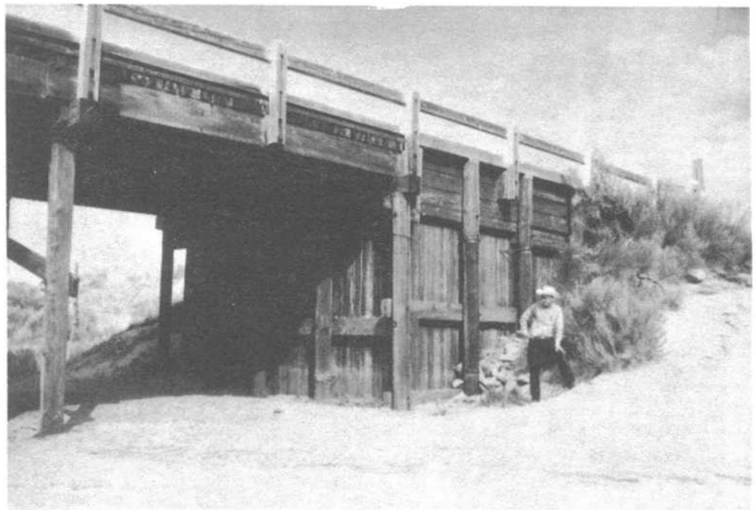

Figure 21. Successful engineering design for combating piping under bridge abutments in area of severe and destructive piping.

beside or above it. A riser would surface on the downslope side of the road to receive the runoff from the culvert under the road. The trench would be wet compacted during back filling, to preclude piping induced by subsidence, and the area around the culvert intake on the upslope side of the road would be asphalted as would a relatively small area around the riser. Although it would be much less expensive to extend the culvert overland to allow it to discharge directly into the gully, this method might result in a structural failure due to piping under the extended conduit.

Asphalt curbs could be emplaced along both sides of the road to prevent roadway runoff from causing piping of highway fills and subjacent terrain. The collected runoff could be led along the side of the roadway via an asphalt-lined shallow drainage way which would discharge into the buried drain of the nearest culvert.

It is believed that grading of nearby arroyo walls to roughly a 1:1 slope would decrease the intensity of piping along the arroyo. However, the reduction in piping may not be sufficient to justify the expense. Grading of the arroyo banks would decrease adjacent piping intensity for two reasons: (a) development of stress-desiccation cracks parallel to the gully wall, such as are shown in Figure 17, would be greatly reduced; and (b) increased velocity of runoff would tend to cause the water to override small cracks as well as to plug them by surface erosion.

Bridge deck and adjacent roadway drainage should be collected and dropped into the channel by means of drains similar to the gutter downspouts on houses. Such a successful device is shown in Figure 20. A different, but more expensive, approach which appears to have been successful is to box the abutment slope with planking (Fig. 21).

Two unsuccessful attempts to arrest and control piping at the Arizona Highway 264 Jeddito Wash bridge on the Navajo Reservation are shown in Figures 22 and 23. As shown in Figure 22, steel fence posts and barbed wire strands were emplaced on a line parallel to and about 2 ft away from the abutment wall. This enclosure was then filled with boulders, but the piping continues unabated and the boulders are now funneling vertically into the ever-enlarging pipe system. To stop this piping, the supply of water from the overlying bridge deck and roadway pavement must be intercepted and led to the arroyo channel in downspout conduits. Grout or slurry pumped into the boulder-filled pipes would help stabilize the abutment slope. Instead of leading bridge-deck drainage to the arroyo channel through conduits at the Jeddito Wash bridge as was done at the Meadow Valley Wash bridge (Fig. 20), shallow concrete troughs were constructed to intercept the drainage at the land-slope surface and conduct it to the

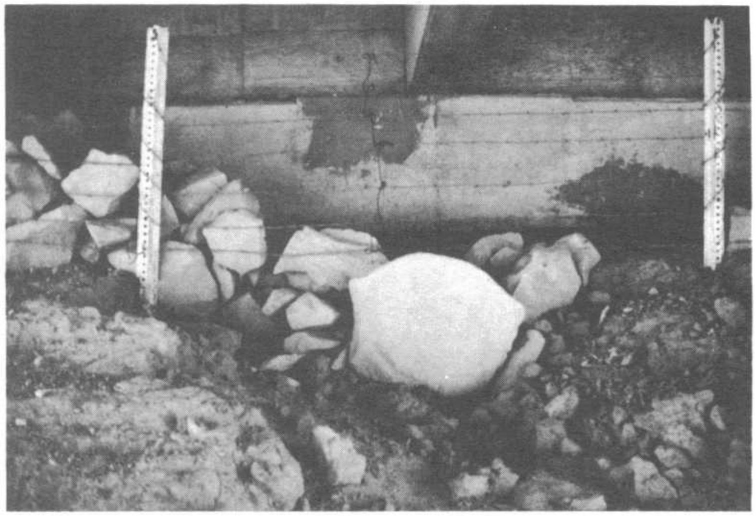

Figure 22. Abutment showing unsuccessful attempt to control piping. Pipes continue to enlarge and boulders fall into pipe inlets.

Figure 23. Unsuccessful use of concrete trough to intercept surface runoff and bridge-deck drainage, and conduct it to arroyo channel. Hole about one foot in diameter has broken through the trough to the underlying pipe. Outlet of pipe system is hidden behind bridge pier in right foreground but is undermining pier.

channel (Fig. 23). Piping continues, however, and a hole has been broken through the trough, probably by livestock crossing the slope. Asphalting the total area enclosed within a 50- to 100-ft area of culvert inlets could be expected to reduce greatly the damage currently being done to culvert installations. Extension of culvert outlets to a nearby gully channel would also greatly decrease the damage being done.

SUMMARY

Piping is most commonly observed in alluvium in arid and semiarid regions. However, it also occurs in loess and in certain agrillaceous bedrock, especially shales and altered volcanic ash and tuff. It frequently results in a pseudokarst topography.

Subsidence, or collapse structure as it is sometimes called, has been found both to initiate and to accompany piping. Three types of piping have been delineated depending on the mode of origin of the pipes: (a) desiccation and stress-desiccation cracks, or sometimes localized subsidence cracks or holes, with visible inlets and outlets; (b) entrainment without visible inlets but often with visible "boils" as outlets; and (c) variable permeability-subsidence, developed along a temporary perched water table, a stratum of relatively high permeability, or subsidence-caused voids with visible outlets but initially without visible inlets.

Our preliminary findings with regard to characterization of strata which are subject to piping, generally corroborate those of Quirk and Schofield (25) and Aitchison, Ingles, and Wood (26). Essentially, silty and clayey earth materials are susceptible to piping if they:

1. Contain in excess of about 20 percent montmorillonite;
2. Desiccate thoroughly, or are susceptible to localized subsidence, or have a stratum of high permeability relative to lower strata, or have a temporary perched water table; and
3. Contain a high percentage of exchangeable sodium. However, due to subsidence, loess and possibly other low-density previously unwetted earth materials may pipe even in the absence of montmorillonite and large amounts of exchangeable sodium.

Regardless of location or materials, at least four basic conditions are essential for piping to develop: (a) sufficient water either to cause drainage through cracks or to saturate a layer of higher permeability than the layers below it, (b) hydraulic head sufficient to move water through a subsurface route, (c) presence of a permeable (or deeply cracked) soil or bedrock above gully floor level, and (d) outlet for flow.

The mechanisms of piping are now largely understood, although details of the physicochemical-mineralogical aspects are still obscure. The geologic formations and geographical areas in the United States which are particularly susceptible to piping can be recognized. Based on the information presented in this report, a careful scrutiny of proposed highway routes for evidence of piping, by competent individuals, could readily avoid the increased cost resulting from the unwise selection of highway routes in areas subject to piping. However, where piping materials must be utilized for road construction, several precautions can be taken to minimize damage to highways by piping. Catchment drains under bridge deck-roadway junctions are essential and use of asphalt-mat covers may also be necessary at some sites. Culvert discharge should be carried in closed conduits to empty into a gully distant from the roadbed. Asphalting the area around culvert inlets would reduce the current damage being done to roads in areas where piping is a problem. The main rules to follow are: (a) to avoid terrain and road-building materials susceptible to piping whenever possible; and (b) to prevent the concentration of runoff, thus, preventing piping near roadways, bridges, or drainage structures.

ACKNOWLEDGMENT

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REFERENCES

1. Parker, Garald G. Piping, a Geomorphic Agent in Landform Development of the Drylands. Internat. Assoc. of Sci. Hydrology, Publ. No. 65, pp. 103-113, 1963.
2. Parker, Garald G., Shown, L. M., and Ratzlaff, K. W. Officer's Cave, a Pseudokarst Feature in Altered Tuff and Volcanic Ash of the John Day Formation in Eastern Oregon. Geological Society of America Bull., Vol. 75, No. 5, pp. 393-402, 1964.
3. Bond, R. M. Rodentless Rodent Erosion. Soil Conservation, Vol. 10, 269, 1941.
4. Carroll, P. H. Soil Piping in Southeastern Arizona. U. S. Department of Agriculture duplicated report, 1949.
5. Cockfield, W. E., and Buckharn, A. F. Sinkhole Erosion in the White Silts of Kamloops. Royal Society of Canada Trans., 3rd Series, Vol. 40, Sec. 4, pp. 1-10, 1946.
6. Downes, R. G. Tunnel Erosion in North-Eastern Victoria. Commonwealth Sci. and Indus. Research Jour., Vol. 19, pp. 283-292, 1946.
7. Gibbs, H. S. Tunnel-Gully Erosion on the Wither Hills, Marlborough. New Zealand Jour. of Set. and Tech., Vol. 27A, No. 2, pp. 135-146, 1945.
8. Aitchison, G. D., and Wood, C. C. Some Interactions of Compaction, Permeability and Post-Construction Deflocculation Affecting the Probability of Piping Failure in Small Earth Dams. Proc., Sixth Internat. Conf. on Soil Mech. and Foundation Eng., Montreal, Vol. 2, pp. 442-446, Univ. of Toronto Press, 1965.
9. Cowan, E. H. Personal communication, June 1966.

10. Bishop, W. W. Gully Erosion in the Queen Elizabeth National Park. Uganda Jour., Vol. 62, No. 2, pp. 161-165, 1962.
11. Jennings, J. E., and Knight, K. Recent Experiences With Consolidation Tests as a Means of Identifying Heaving or Collapse of Foundations on Partially Saturated Soils. Trans., Southern African Inst. of Civil Eng., Vol. 6, No. 8, pp. 255-256, 1956.
12. Jennings, J. E., and Knight, K. The Additional Settlement of Foundations Due to Collapse of Structure of Sandy Subsoils on Wetting. Proc., Fourth Internat. Conf. on Soil Mech. and Foundation Eng., London, Vol. 1, pp. 316-319, 1957.
13. Knight, K. The Microscopic Study of the Structure of Collapsing Sands. Proc. Second Southern African Regional Conf. on Soil Mech. and Foundation Eng., Lourenco, Marques, pp. 69-72, 1959.
14. Hardy, Robert M. Construction Problems in Silty Soils. The Engineering Jour. (Montreal), Vol. 33, pp. 775-779 and 782, 1950.
15. Poland, J. F. Land Subsidence in the San Joaquin Valley, California, and Its Effect on Estimates of Ground Water Resources. Internat. Assoc. of Sci. Hydrology, Publ. No. 52, pp. 324-335, 1960.
16. Jumikis A. R. Soil Mechanics. Van Nostrand Co., 1962, 336 pp.
17. Terzaghi, Karl, and Peck, R. B. Soil Mechanics in Engineering Practice. John Wiley Son, pp. 230-232 and 502-514, 1948.
18. Mielenz, R. C., and King, M. E. Physical-Chemical Properties and Engineering Performance of Clays. Calif. Div. of Mines, Bull. 169, pp. 196-254, 1955.
19. Downes, R. G. Conservation Problems on Solodic Soils in the State of Victoria (Australia). Jour. of Soil and Water Conservation, Vol. 11, pp. 228-232, 1956.
20. Brown, George W. Piping Erosion in Colorado. Jour. of Soil and Water Conservation, Vol. 17, pp. 220-222, 1962.
21. Turnbull, W. J. Utility of Loess as a Construction Material. Proc., Second Internat. Conf. on Soil Mech. and Foundation Eng., Rotterdam, Vol. 5, pp. 97-103, 1948.
22. Lowry, M. E. Personal communication, April 1964.
23. Hadley, R. F. Personal communication, Feb. 1963.
24. Buckharn, A. F., and Cockfield, W. E. Gullies Formed by Sinking of the Ground. Amer. Jour. of Sci., Vol. 248, pp. 137-141, 1950.
25. Quirk, J. P., and Schofield, R. K. The Effect of Electrolyte Concentration on Soil Permeability. Jour. of Soil Sci., Vol. 6, pp. 163-178, 1955.
26. Aitchison, G. D., Ingles, O. G., and Wood, C. C. Post-Construction Deflocculation as a Contributory Factor in the Failure of Earth Dams. Proc. Fourth Australia-New Zealand Conf. on Soil Mech. and Foundation Eng., pp. 275-279, 1964.

Steady State Drainage Flow in Peat

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● THE drainage of cohesive soils is important in road and agricultural engineering. Information is required for design purposes on problems such as optimum location of longitudinal side-drains, profile of phreatic water surface, change in water content and degree of saturation brought about by drainage, and duration of unsteady phase of groundwater lowering.

This paper describes investigations undertaken on the flow of water to drains in peatland, and is based on the results of full-scale experiments carried out in the field. The vast majority of papers published on this topic are based on experiments involving model studies of flow in sand tanks, electrical analogs, membrane analogies, or analytical methods comprising mathematical or numerical analysis. The most generally used drainage device in the reported work is a well. A well was also used in our experiments, as accurate control of discharge is best effected by this device. The equations governing radial flow to wells are readily adapted for the conditions of parallel flow to trenches (2).

Flow into wells can be divided into two classes: (a) confined flow in which the water is confined under pressure in a stratum sandwiched between two impermeable layers giving rise to artesian conditions; and (b) unconfined or gravity flow in which the water-bearing stratum is not confined by an upper impermeable layer. In this case the groundwater is exposed to the atmosphere, and an imaginary boundary known as the phreatic surface (at all points of which the fluid pressure is atmospheric) constitutes the upper surface of flow, if capillary flow is neglected. All pumped wells are initially in a transient condition. However, flow eventually changes from the nonsteady to the steady state. This can be assumed to have been achieved when no appreciable variation occurs in the water table at a reasonable distance from the pumped well.

The results measured in these experiments are compared with results calculated from formulas proposed by Dupuit (1), Jaeger (2), Babbit and Caldwell (3), Hansen (4), Boulton (5), and Hall (6).

NOTATION

A	=	cross-sectional area through which flow takes place (Darcy formula)
c	=	density of fluid
C_x	=	a coefficient defined by a curve relating drawdown to radius (Babbitt and Caldwell equation)
g	=	acceleration due to gravity
H	=	height of undisturbed water surface above the bottom of the well
h_o	=	depth of water in well during pumping
h_s	=	height of the free surface above the bottom of the well at the well edge
h_s-h_o	=	surface of seepage
h_{115}	=	height of the free surface above the bottom of the well at a distance of 115 r_o from the well center
h	=	height of phreatic surface above the bottom of the well

Paper sponsored by Committee on Embankments and Earth Slopes and presented at the 46th Annual Meeting.

$\bar{h}$ = height of liquid column (Forchheimer equation)
i = hydraulic gradient
K = a constant depending on the medium and known as the coefficient of permeability
ln = natural logarithm or logarithm to the base e
log = logarithm to base 10
p = penetration expressed as a fraction of the full depth of the water-bearing stratum
Q = volume of water discharged in unit time
R = radius of influence or the radial distance from the well at which the water level is not affected by pumping
r_o = radius of the well
r = radial distance from the well
x, y = coordinates of point h projected on the horizontal plane
μ = coefficient of viscosity
θ = slope of the free water surface at point (r, h)

PREVIOUS INVESTIGATIONS

Hall (7) has reviewed in detail the development of knowledge on the theory of seepage toward wells. Many other workers have carried out extensive analyses on flow through porous media. The results of these investigations in so far as they apply to the case of steady flow to a gravity well will be briefly summarized.

In 1856, Darcy (8) proposed a relationship, now known as Darcy's law, which has become the basic equation for the flow of water through a saturated porous medium. The equation may be written as follows:

$$Q = KAi \tag{1}$$

This basic equation is valid in the case of laminar flows only. The critical value of the Reynolds number, at which the flow through soil changes from laminar to turbulent, lies between 1 and 12. In the present study of flow through peat, the velocity of flow and the effective particle size diameter are so small that the Reynolds number is very much less than one and Darcy's law can be taken as valid.

Dupuit (1) assumed that for small inclinations of the free surface of a gravity-flow system, the streamlines (a) can be taken as horizontal, and (b) are associated with a velocity proportional to the slope of the free surface but are independent of depth. From these assumptions he derived the Dupuit equation for radial flow to a well:

$$Q = \frac{\pi K (H^2 - h_o^2)}{\ln R/r_o} \tag{2}$$

Forchheimer (9) applied the Dupuit assumptions combined with the equations of continuity to the analysis of a fluid in any column of liquid of height $\bar{h}$ above an impermeable base of a layer through which flow is taking place. His treatment yielded the following general equation of the free surface in gravity-flow system,

$$\frac{\delta^2 \bar{h}^2}{\delta x^2} + \frac{\delta^2 \bar{h}^2}{\delta y^2} = 0 \tag{3}$$

This result involving the function $\bar{h}^2$ is analogous to that which is obtained when Darcy's law is combined with the continuity equations, from which operation the function h (total head) is also found to satisfy the Laplace equation

$$\frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta y^2} = 0 \tag{4}$$

Boulton (5) determined the free surface by the relaxation method. He also showed that the discharge given by the Dupuit formula was within 1 percent of the actual discharge as measured on a sand model.

Muskat (10) analyzed the flow to wells on a purely theoretical basis and with Wyckoff and Botset carried out a number of sand tank experiments. The sand tank used was equipped with manometers connected to the base. These measured the radial pressure distribution above the bed. The fluid in the tank was kept in continuous circulation so that steady state conditions were easily maintained. They found that the following formula, after correcting for flow in the capillary zone, gave very accurate measurements of discharge:

$$Q = \frac{K\pi\, cg\,(H^2 - h_0^2)}{\mu \ln R/r_0} \tag{5}$$

They also concluded that base piezometric heads rather than heads due to the free surface were given by the Dupuit equation.

Babbitt and Caldwell (3) carried out extensive analyses at the Engineering Experiment Station of the University of Illinois. Their object was to formulate the conditions of flow into gravity wells so that greater precision in the solution of problems could be achieved. They used electric models and sand tanks during the experiments. The Dupuit formula gave accurate results, provided the ratio of well drawdown to the thickness of the groundwater stream penetrated was less than 0.2. On investigation, they noted that the area of influence was circular in very special cases only. It becomes elliptical where the water table is sloping. However, the error involved in assuming the region of influence to be a circle is negligible and the radius of such a circle may be substituted into the Dupuit formula. Kozeny's equation

$$Q = \frac{\pi K (H^2 - h_0^2)}{\ln R/r_0} \left[1 + 7\sqrt{\frac{r_0}{2H}} \cos \frac{\pi p}{2} \right] \tag{6}$$

for calculating the discharge of a partially penetrating well was checked and found to be accurate.

They introduced a new equation for the determination of the free surface

$$Q = \frac{\pi KH (H - h)}{C_x \ln R/0.1\,H} \tag{7}$$

This equation was based on observations of the free surface in sand tank and electric analog tests, and though empirically derived gave very accurate results.

Hansen (4) carried out experiments using membrane analogy and sand tanks. He verified Muskat's findings that the Dupuit equation gave the piezometric heads above the base rather than the free surface. Having studied the previous work of Babbitt and Caldwell (3), he then proposed another slightly different equation for the free surface by substituting 0.3 log R/r for C_x:

$$Q = \frac{\pi KH (H - h)}{0.3 \log R/r \ln R/0.1\,H} = \frac{\pi KH (H - h)}{0.69 \log R/r \log R/0.1\,H} \tag{8}$$

Hall (6) conducted a series of large-scale tests with a sand tank and found that the Dupuit equation gave base piezometric rather than free surface heads. He also compared the flow patterns calculated by means of Yang's relaxation technique with the actual flow pattern observed in the tests and found that the maximum discrepancy was of the order of 6 percent. Finally, he proposed two empirical equations for calculation of the free surface close to the well:

$$\frac{h_s - h_o}{H - h_o} = \frac{1 - (h_o/H)^{2.4}}{\left[1 + \frac{1}{50}\ln R/r_o\right]\left[1 + 5/H/r_o\right]} \tag{9}$$

$$\frac{h - h_s}{H - h_s} = 2.5\left(\frac{r - r_o}{R - r_o}\right) - 1.5\left(\frac{r - r_o}{R - r_o}\right)^{1.6} \tag{10}$$

Many workers had questioned the validity of the Dupuit equation. Its analytical correctness has however been verified by Polubarinova-Kochina (11), Chapman (12, 13, 14) and more recently by Hantush (15). The only limitation imposed by Hantush was that the data analyzed should be from wells at radius $r > 1.5h$ from the pumped well.

Jaeger (2) stated that because of the simplifying assumptions introduced, the Dupuit formula was valid only as an approximation. Furthermore, it should only be used where the value of θ was small. Dupuit's assumption that along any vertical line the streamlines are horizontal in which case the equipotentials are vertical can lead to great errors if used close to the well. He suggested that only a small error was involved in assuming that the equipotentials were circular arcs perpendicular at one end to the impermeable base and at the other end to the water table at any distance from the well. From this hypothesis he developed his equation for flow to a well:

$$Q = 2\pi Krh\,(1 + h/2r \tan \theta/2)\,\theta \tag{11}$$

An examination shows that this equation cannot be solved by direct integration, The shape of the water table can however be determined from one known point by the method of finite differences. As this is only an approximation, the best results are obtained by taking very small increments of r. This applies, especially, near the well where the value of θ changes rapidly. The amount of work involved in deriving the water table curve in this manner is very great. However, the repetitive nature of the calculations lends itself to analysis by digital computer.

Surface of Seepage

The Dupuit equation is generally used in connection with flow to gravity wells. However, because of the assumptions made by Dupuit that the streamlines are horizontal, the equation will be most accurate at large radial distances from the well where the free surface approaches the horizontal. The greatest inaccuracy in the Dupuit analysis, however, is that the surface given by the equation intersects the well at the level of water in the well. It has been established that this free surface intersects the well face at a point above the water level, giving rise to what is known as the surface of seepage along the well face. When the drawdown is small the surface of seepage is also small but as the drawdown increases, particularly in the case of gravity flow where the ratio of drawdown to total depth can be very large, the water level close to the well will be considerably higher than that given by the Dupuit equation.

Harr (16) states that the first approximate method that accounts for the development of the surface of seepage was proposed independently in 1916 by Schaffernak (17) and van Iterson (18). Muskat (10) showed that the surface of seepage must exist as otherwise there would be an infinite velocity at the point of intersection of the water table and well wall. He further stated that since Dupuit had based his conclusions on erroneous assumptions and had neglected to take account of the surface of seepage, his equation should automatically fail. However, results have shown that when the measuring well is far enough removed from the pumped well, the Dupuit equation gave surprisingly accurate results. Muskat therefore concluded that the Dupuit equation, despite the erroneous assumptions, could be used to give accurate results of the discharge, although he was of the opinion that these results were entirely fortuitous. Babbitt and Caldwell (3), Hall (6) and others recognized the importance of the surface of seepage and proposed empirical equations (7, 9, 10) to measure its height. Hansen (4) was of the opinion that the use of the radius of influence could lead to inaccurate results as it is rather an indefinite and hypothetical measurement. He formulated an expression that used only functions measurable at the well face. By combining these functions in a number of dimensionless parameters he derived the equation:

$$Q/Kr_o^2 = f_1\,(h_s/r_o,\ h_o/r_o) \tag{12}$$

Using the results from the electric analog tests of Babbitt and Caldwell and from his own sand tank measurements, Hansen developed a family of curves based on these dimensionless parameters. These curves can be used to measure the height of the surface of seepage without reference to the radius of influence.

Zee, Peterson and Dock (19) did some further dimensional analysis and introduced another dimensionless parameter and a new equation:

$$Q/K\,r_o^2 = f_2\,(h_s/r_o,\ h_o/r_o,\ h_{115}/r_o) \tag{13}$$

which can also be used to find the height of the surface of seepage.

Boulton (5) set out specifically to devise a means of measuring the free surface and the surface of seepage. Using relaxation methods, he produced curves by which these could be calculated.

An examination of Jaeger's equation shows that the higher the value of Q, the steeper the water table curve becomes. For small values of Q, the curves intersect the well surface at the level of water in the well. At a particular discharge, the water table curve becomes tangential to the well face. Jaeger postulated that the discharge producing this curve was a critical discharge (Q_c) and that the corresponding depth of water in the well was a critical depth (h_c). Larger discharges produce curves that do not reach the well face and represent imaginary rather than real flows. Lowering the water level in the well below the critical depth therefore would produce a water table appropriate to the critical depth and a surface of seepage would develop between the critical depth and the water level in the well.

In summary, Dupuit completely neglected any consideration of a surface of seepage. Jaeger suggested that a critical depth existed. Where the water level in the well is higher than the critical depth, no surface of seepage occurs. However, if the water level in the well is lower than the critical depth, a surface of seepage develops over this range. Most other workers including Babbitt and Caldwell, Hansen, Boulton, Hall, and Muskat are agreed that a surface of seepage develops immediately after the water level in a well is lowered and that it increases as the water level is further lowered.

FIELD INVESTIGATIONS, FIRST WELL EXPERIMENT

Location

The field work was carried out at the Peatland Experimental Station of the Agricultural Institute, Glenamoy, County Mayo. The Station is situated in an extensive area of blanket peat which varies in depth from 4 ft to 25 ft and averages about 14 ft. The peat is very gelatinous and has a high capacity for water retention. The moisture content is usually around 1400 percent. The water table fluctuates from a few inches below the surface in winter to about 9 in. down in summer.

Experimental

A general view of the experimental site is shown in Figure 1. A 6-in. diameter well made from galvanized wire-mesh gauze was positioned in the center of the experimental area. An area of 60 ft x 60 ft surrounding the well was covered with heavy-duty black polythene sheeting to minimize the effects of rainfall and evapotranspiration in the immediate vicinity of the well.

Three lines of pipes radiating from the central well were installed for the purpose of checking on water table fluctuations. On each line, pipes were driven solidly into the underlying marl at 1-ft, 2-ft, 4-ft, 8-ft, 16-ft, and 32-ft distances from the center of the well. The pipes used were ¾-in. OD 18 SWG conduit 16 ft long. Each pipe was drilled with ⅛-in. diameter holes at 1-in. centers and was closed at its lower end. An effective seal against the ingress of water at the junction of each pipe and polythene was provided. Shallow trenches 8 in. wide x 4 in. deep were excavated across the site 20 ft apart. The polythene was fixed into position in these trenches. In this manner surface water drainage was provided and the danger of wind damage to the polythene obviated. A rubber stopper on top of each pipe prevented rainfall infiltration. The water level in the well was controlled by a float attached to a magnetic type switch which actuated a 1/6 bhp totally enclosed electric motor. This was coupled through a reduction gear to a stainless-steel pump and gave discharges of about 12 gph. It was decided to control the water in the well at a number of predetermined levels. To accomplish this, a tolerance of ½ in. above and below the predetermined water level was allowed. In practice, this meant that the pump cut in when the water level rose ½ in. above the required level and worked until the level dropped to ½ in. below. Under normal conditions approximately 0.2 gallon was pumped at about 1-hour intervals.

A ½-in. diameter brass suction pipe was fixed to the well base and had an inlet at 1 ft above the bottom of the well. The float arm was made from narrow bore P. V. C. tubing and could readily be extended or shortened. A 150-watt bulb mounted beside the pump gave sufficient heat to prevent icing during winter.

The pump and switch were mounted on a heavy base which was positioned beside the well. They were protected by an aluminium cover which also covered the well. This accomplished the dual purpose of excluding rain and of allowing the float arm to operate under very stable conditions even in gale-force winds. The pumping unit is shown in Figure 2.

The discharge from the pump was delivered through ½-in. diameter tubing, lagged with glass fiber, to a collecting tank. It was pumped through a perspex float chamber mounted on the roof of the tank (Fig. 3). The float operated a pen on a daily chart and recorded the time of each pumping operation. The water in the collecting tank was

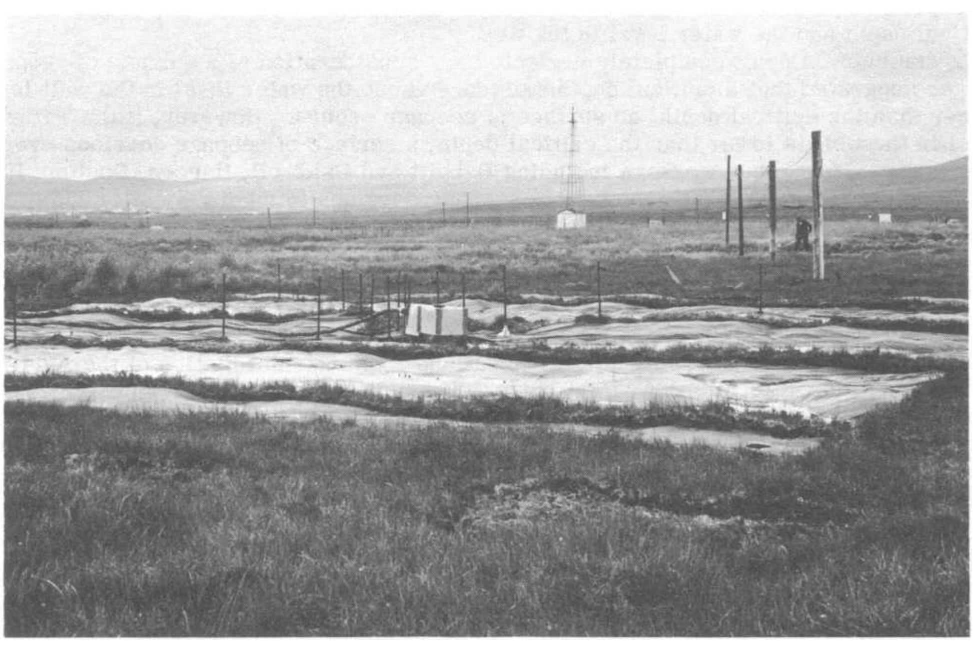

Figure 1. General view of experimental area.

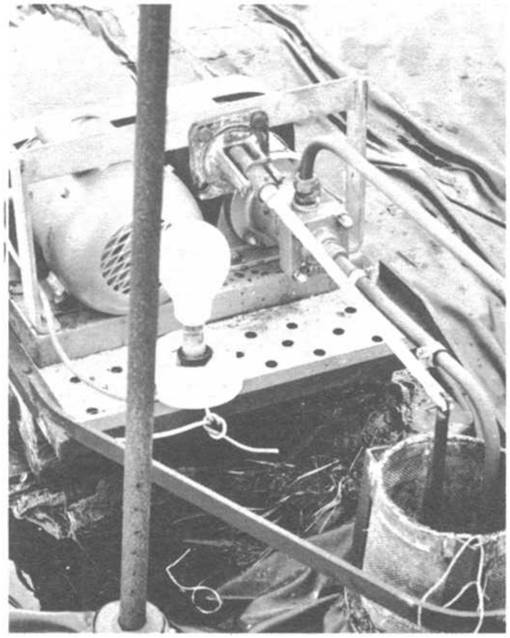

Figure 2. Pumping unit and central well.

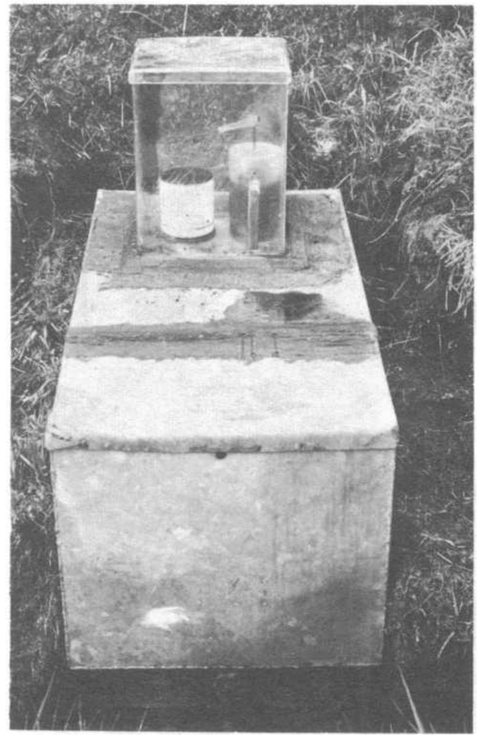

Figure 3. Collecting tank and float chamber.

measured and emptied daily. The installation of the float chamber had two objectives: (a) to check on the regular operation of the pump–any miss in the pumping cycle could indicate a sticking float or some other mechanical or electrical breakdown; and (b) to enable the actual daily discharge to be accurately calculated. At a meteorological station beside the site, the usual meteorological data including rainfall, temperature, sunshine hours, wind run and evaporation were recorded daily.

The water table level in each pipe was measured by means of a battery operated probe 6-ft. long (Fig. 4). This was made from ½-in. diameter copper tubing graduated in 0.01-ft intervals attached to an indicating unit consisting of a milliammeter, 1½-volt battery, resistance and switch. The top of each water table pipe was leveled from a temporary bench mark. These levels were checked at intervals throughout the course of the experiment–the variation was negligible in all cases.

Measurements

When all installations were complete, the water table levels in the central well and pipes were measured for a few days until stable conditions were obtained. Then the water level in the central well was lowered approximately 1 ft, and the float was set to control it at this depth. The discharge and water table levels were measured daily. After a period, the water level in the central well was lowered to approximately 3 ft, then to 5 ft and finally to 7 ft. Measurements continued during each well drawdown after which the float was removed and the area allowed to recharge.

The results of each pumping cycle followed roughly the same pattern. A comparatively large daily discharge occurred on first lowering the water level in the central well. However, as the cone of depression extended, the discharge rate fell off and eventually became almost constant. The water levels in the pipes behaved in a somewhat similar fashion. After a few days, large depressions in the water levels were

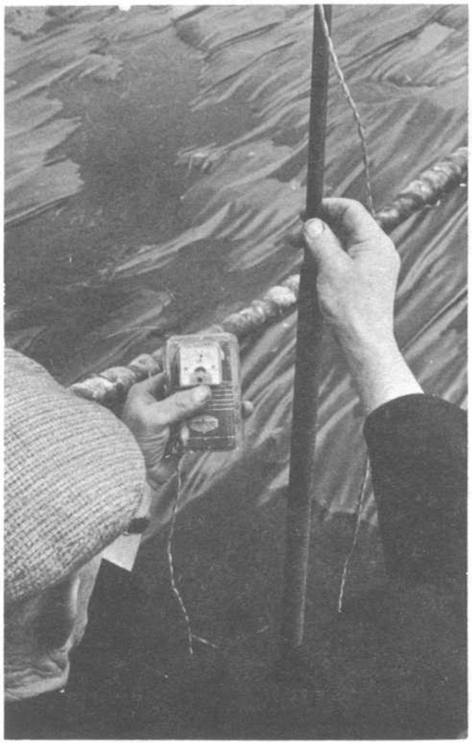

Figure 4. Water level measurement–close-up of indicating unit.

TABLE 1
MEASURED DRAWDOWNS ALONG THE EAST LINE UNDER STEADY STATE CONDITIONS FOR DIFFERENT WELL LEVELS

Well Drawdown (ft)	Radial Distance (ft)					
	1	2	4	8	16	32
0.97	0.66	0.51	0.38	0.26	0.16	0.07
2.94	1.73	1.25	0.94	0.60	0.34	0.08
5.03	2.03	1.55	1.18	0.79	0.49	0.18
7.03	2.19	1.72	1.31	0.90	0.57	0.27

TABLE 2
DAILY DISCHARGE FIGURES UNDER STEADY STATE CONDITIONS[a]

Well Drawdown (ft)	Measured Discharge (cc per day)	Corrected Discharge (cc per day)	Permeability (cm per day)
0.97	12,816	10,777	0.9816
2.94	24,804	20,857	0.6625
5.03	21,859	18,381	0.3911
7.03	19,911	16,743	0.2937

[a]Corrected for partial penetration of the well, and corresponding permeabilities calculated for the East line by the Dupuit formula.

TABLE 3
AVERAGE TOTAL CONSOLIDATION MEASURED AT THE CONCLUSION OF EACH PUMPING CYCLE

Well Drawdown (ft)	Radial Distance (ft)			
	1-4	8	16	32
0.97	0.04	0.02	–	–
2.94	0.10	0.04	0.02	–
5.03	0.14	0.07	0.03	–
7.03	0.18	0.10	0.05	0.03

observed in the pipes nearest the central well. The rate of fall in these pipes was gradually reduced and the influence of the central well on the water levels in the other pipes became more evident. In time, the water table variations practically ceased and the drawdown curve became stable. Thus, the steady state daily discharge rate and the corresponding phreatic surface were measured and recorded for each pumping cycle. During the course of the experiment the general groundwater table in the area fluctuated due to rainfall and evapotranspiration. These fluctuations gave rise to occasional variations in the measured water table levels. Heavy rainfall caused sudden increases in the water table measuring pipes, but after a few days the levels again dropped to the original.

The chart recorder on the discharge line proved most useful in measuring the actual daily discharge from the well. The total discharge was measured and a new chart mounted each morning. However, the measured discharge did not represent a 24-hour figure as the pump scarcely ever cut in at the same time on two successive mornings. By reference to the charts, the actual time over which the measured discharge occurred was computed and the measured quantity adjusted to give a 24-hour discharge figure. This correction was necessary to establish whether or not a steady state condition had been achieved and to measure the actual steady daily discharge.

Results

As the experiment progressed, it became apparent that there was a general groundwater movement across the experimental area. This caused occasional slight fluctuations of the water levels in some observation pipes. Pipes along the East line were least affected by these fluctuations and the discussion is based on measurements made on that line. The drawdown figures under steady state conditions for each well lowering are given in Table 1 and the corresponding measured discharges in Table 2.

Correction for Partial Penetration

The drainage well did not penetrate the full depth of peat to the top of the impermeable layer. The measured discharge was therefore increased by a contributory flow from the peat below the bottom of the well. However, as already stated, Kozeny had postulated and Babbitt and Caldwell had proved that Eq. 6 could be used to calculate the correct discharge from a partially penetrating well. The steady state discharge figures were corrected by this formula and are given in Table 2.

Dupuit Formula

The Dupuit formula (Eq. 2) was used to calculate the permeability of the peat during each pumping cycle. Many workers have verified that this formula can be used to give accurate results, provided the observations are made at a point far enough removed from the central well to lie in the Dupuit zone. Peterson (20) has stated that the observations should be made at a minimum distance of 100 r_o (25 ft) from the central well while Hantush (15) recommends that at the point of observation r should exceed 1.5h. The 32-ft drawdown figures were used in the calculations.

This radial distance more than satisfied the requirements. The calculated permeabilities (centimeters per day) are given in Table 2. Permeability tests were also run in the laboratory using a modified Bjerrum-Huder apparatus (23). These tests will be described in detail. However, it can be noted here that the permeability figures resulting from the laboratory tests all lay between 0.31 cm per day and 1.52 cm per day. The 0.31 cm per day was achieved while the pore water of the sample was under a suction of -4 psi while the 1.52 cm per day resulted with the pore water under a positive pressure of 12 psi. These measurements check in very well with the field results which run from 0.29 cm per day to 0.98 cm per day.

Permeability and Discharge Variations

Table 2 shows a number of apparent anomalies. One would normally expect that the permeability figures would remain more or less constant throughout the four pumping cycles. An increasing discharge would also be expected as the pumping depth was increased. This discharge might become constant at some stage but would not be expected to decrease. However, reference to the table shows that the maximum permeability was achieved at the 0.97-ft drawdown level. Each succeeding pumping cycle gave a smaller permeability, culminating in the minimum figure at the 7.03-ft well drawdown. The expected increased flow between the first and second pumping cycles occurred. However, this increase was not maintained and the next two cycles produced decreasing flows.

The decrease in permeability, amounting to about 30 percent, that occurred between the first two pumping cycles during which time the discharge rate almost doubled itself is most probably explained by the fact that the top 18 in. of peat is more open and fibrous than the remainder. During the first pumping cycle the water table lay in this zone, but for the other cycles most of the flow was through the gelatinous peat. The further reductions calculated for the 5.03-ft and 7.03-ft well drawdowns are directly related to the decreased discharge rates. These reductions in measured discharge were most unexpected. Normally one expects the discharge to increase with increasing depth or, presupposing the validity of Jaeger's theory, to increase to a maximum and remain constant at that maximum value. However, as an examination of Table 2 shows, the decrease was evident right through each cycle. An explanation was sought.

Ground Level Variation

The consolidation of peat due to the groundwater lowering during each pumping cycle was measured. Fixed points on the surface were checked from a temporary bench mark after each cycle. These points were positioned beside the water table observation pipes so that 18 levels were taken on each occasion. The results of these measurements are given in Table 3. There was a comparatively large amount of traffic in the immediate vicinity of the well. On this account, practically identical consolidation figures

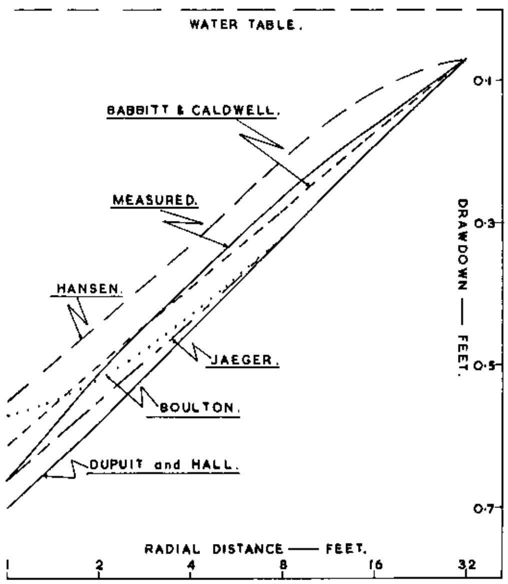

Figure 5. Semilog plot of measured and calculated drawdown curves for 0.97-ft well drawdown.

were achieved at the 1-ft, 2-ft, and 4-ft, distances. For convenience, these figures are bulked in the table under the heading 1 ft to 4 ft.

When one considers that the total depth of peat at the site of the experiment is about 13.3 ft, it becomes obvious that consolidation alone could not have accounted for the large decreases in permeability and discharge.

Entrapped Air

Another factor which could have had an adverse effect on the permeability was entrapped air. Groundwater holds a quantity of air in solution. This, if released, could occupy a percentage of the pore space and cause a reduction in permeability.

The general water table level close to the well was reduced during pumping. The water in the peat above this reduced level was consequently subjected to tension. Similarly, the water in the peat at all depths below the new water table was under reduced pressures. The general effect, therefore, on the groundwater was one of reduced pressures with tensile forces coming into play above the new water table. This effect was most pronounced at the well face and decreased as the distance from the well increased. These considerations suggested that air could come out of solution and clog pore space, especially at points close to the face of the well. A separate laboratory experiment was undertaken to investigate this. The results showed that as the pore water pressure in the peat was reduced, the measured permeability also dropped. More recently, a paper by Orlob and Radhakrishna (22) has come to hand in which the effects of entrapped gases on permeability are discussed. Their experiments showed that a 10 percent increase in air content could cause a reduction of 35 percent in permeability. This confirmation of a theory that we had independently suggested and proved was most encouraging. It leaves us in no doubt that the decreased discharge rates and the correspondingly reduced permeabilities were caused by entrapped air which was released from solution as the water table was lowered.

Discussion

Babbitt and Caldwell, Hansen, Hall and Boulton all agreed that the Dupuit curve gave base piezometric rather than free surface levels. This, in effect, means that at points sufficiently far removed from the well to lie in the Dupuit zone, the Dupuit formula gives actual drawdown levels. At points inside the Dupuit zone, the actual water level is higher than that given by the Dupuit formula. Their proposed equations were designed to give free surface drawdowns close to the well and to merge with the Dupuit curve in the Dupuit zone. These equations were used to calculate free surface curves for different drawdowns. Jaeger suggested that his own equation could be used to give the drawdown curve at all distances from the well without referring to the Dupuit formula. A computer program (23) was written to solve the Jaeger equation by the method of finite differences. The measured and calculated drawdown curves for each pumping cycle are illustrated (Figs. 5, 6, 7, 8). Examination of Figure 5 (0.97-ft well drawdown) shows that the Dupuit, Hall, Jaeger and Boulton curves lie very close together; in fact, the Dupuit and Hall equations give the same curve. The Jaeger and Boulton curves coincide with this curve to the 8-ft mark and diverge slightly from there. The measured drawdown curve lies between the Hansen and Babbitt and Caldwell curves. It approximates very closely to the Babbitt and Caldwell curve–the divergence being of the order of 0.01 it from 32- to 2-ft radial distance.

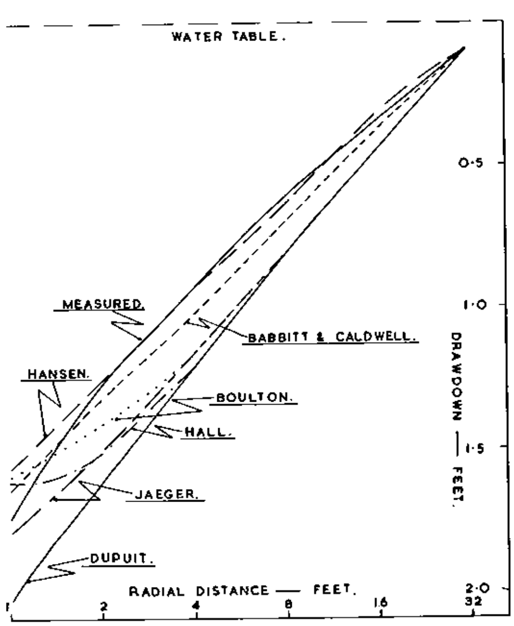

Figure 6. Semilog plot of measured and calculated drawdown curves for 2.94-ft well drawdown.

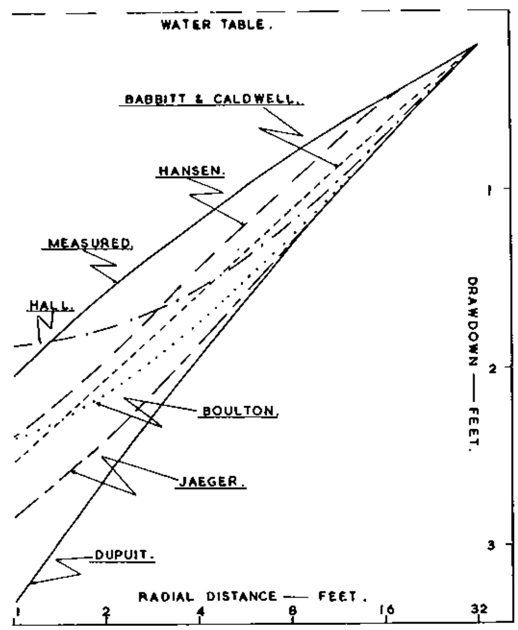

Figure7. Semilog plot of measured and calculated drawdown curves for 5.03-ft well drawdown.

The 2.94-ft well drawdown (Fig. 6) shows a similar pattern. The Dupuit and Hall curves coincide to the 4-ft mark, from which point they diverge. The Boulton curve also coincides with the Dupuit curve as far as the 8-ft mark. The Jaeger and Dupuit curves coincide to the 16-ft mark. They diverge at this point but the divergence is very gradual. The measured drawdown, Hansen and Babbitt and Caldwell curves again lie very close together. The Hansen curve corresponds most closely to the measured drawdown curve. It coincides with it from 2- to 4-ft radial distance and has a maximum divergence of 0.03 ft from 4- to 32-ft radial distance. The Babbitt and Caldwell curve lies below the measured drawdown, the divergence averaging about 0.06 ft. In the 5.03-ft well drawdown (Fig. 7), the same pattern emerges. The Dupuit and Jaeger curves lie close together. The Boulton curve coincides with the Dupuit curve for a time but then diverges rather rapidly. The Hall curve also follows the Dupuit trend but diverges rapidly inside the 8-ft radial distance. The Babbitt and Caldwell curve lies approximately midway between the measured drawdown and Dupuit curves. The Hansen curve again gives the closest approximation to the measured drawdown. However, in this case the divergence increases from 0.03 ft at 16-ft radial distance to 0.34 ft at 1-ft radial distance.

In the 7.03-ft well drawdown (Fig. 8), the Dupuit and Jaeger curves again lie close together. The measured drawdown corresponds with the Hansen curve to the 16-ft mark but then they diverge rapidly. The Boulton, Hall, and Babbitt and Caldwell curves lie approximately midway between the measured and Dupuit curves. Each curve follows its own course, however, and no two curves, except the Dupuit and Jaeger curves, show a similar trend.

An outstanding feature was the good correlation obtained for all pumping cycles between the Jaeger and Dupuit curves. They diverged slightly as they neared the well face but the divergence, in all cases, was of a comparatively minor nature. The Jaeger curve did not, however, give a good correlation with the measured drawdown curve for any cycle, and our experiments indicated that curves calculated from the Hansen and Babbitt and Caldwell equations approximate much more closely to the measured drawdown curve than does the Jaeger curve. In fact, the correlation between the measured drawdown curve and those calculated from the Hansen and Babbitt and Caldwell equations

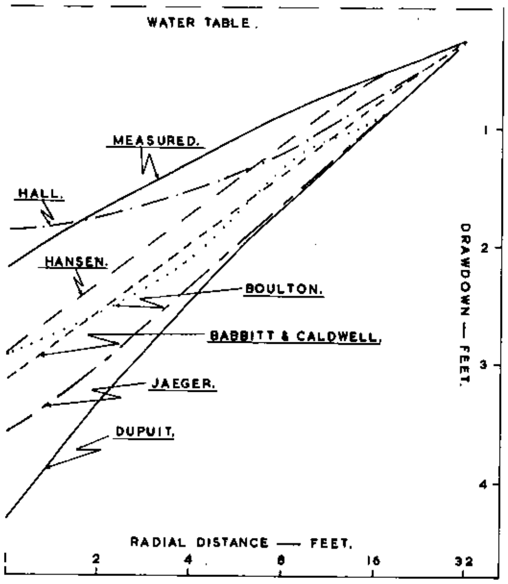

Figure 8. Semilog plot of measured and calculated drawdown curves for 7.03-ft well drawdown.

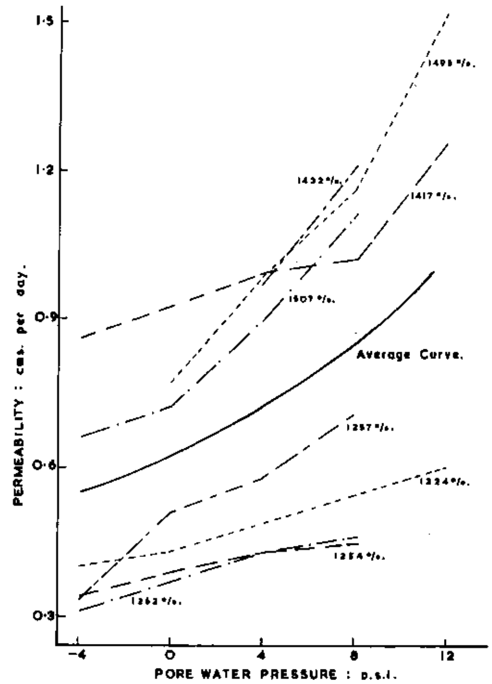

Figure 9. Results of laboratory permeability tests on peat samples showing the variation of permeability at different pore water pressures. Percentage figures quoted are the moisture contents of samples tested.

was excellent for the first two pumping cycles. During the other pumping cycles, however, the measured drawdown curves were affected by air released from solution and trapped within the pore space. This resulted in poor correlation between measured and calculated curves for these cycles.

The close correspondence between the Jaeger and Dupuit curves is as expected when one considers that Jaeger set about correcting what he assumed were errors in the basic Dupuit assumptions but did not take the surface of seepage into account. The experimental results show that a surface of seepage does exist. They also vindicate the accuracy of the Hansen and Babbitt and Caldwell equations and indicate, that for this gelatinous blanket peat, shallow drains (3- to 3½-ft deep) give much better drainage results than deeper drains.

LABORATORY INVESTIGATIONS, PERMEABILITY

When the third and fourth pumping cycles gave reduced steady state discharges, the theory of entrapped air resulting in reduced permeability suggested itself. A comprehensive permeability analysis was then carried out.

The Bjerrum-Huder permeability apparatus (21) consists essentially of a closed system of which the sample is part. This system can be pressurized to any required value by means of a pressure pump, while at the same time a hydraulic head can be generated within the system by raising a mercury column over a system of pulleys. A series of tests was run on a modified Bjerrum-Huder apparatus (23). The peat samples were taken in thin-wall tubes close to the site of the experiment and each sample was tested with its pore water pressure controlled at 0, 4, 8, 12, and -4 psi.

Results

The test results are shown in Figure 9. They fall into two distinct groups. The permeabilities of the first group run from 0.31 cm per day to 0.71 cm per day. The final moisture content of the samples in this group is about 1250 percent. The permeabilities of the second group run from 0.66 cm per day to 1.52 cm per day at final moisture contents of about 1450 percent. All these samples were taken from the same site and at the same depth. We noted however that the final ovendry weights of samples in the lower range were greater than those of the high-range samples. This accounted for the variation in moisture content as a difference of less than 1 grm. dry weight was sufficient to give a moisture content difference of 200 percent. It also suggested that the low-range samples contained more fibrous material than the other samples, and furnished further evidence of the variability of peat and the difficulty of obtaining representative samples.

Discussion

An average permeability curve is shown in Figure 9. This curve represents the average of all laboratory permeability tests carried out and has been weighted to allow for missing values. It shows a permeability range from 0.55 cm per day at -4 psi to 1 cm per day at 12 psi. This corresponds very well with the field values which range from 0.29 cm. per day to 0.98 cm per day. The average laboratory result at 0 psi of 0.63 cm per day compares very favorably with the field result of 0.66 cm per day measured during the 2.94-ft well drawdown.

It is very difficult to represent the result of a lowered water table with its consequent reduced positive pressures and induced suction in a laboratory test. However, our tests do show a very positive reduction in permeability as the pore water pressure is reduced, They also give excellent correlation with field measurements, and we are quite satisfied that entrapped air, released from solution as the water table is lowered, is responsible for the observed reductions in permeability and discharge.

FIELD INVESTIGATIONS, SECOND WELL EXPERIMENT

The pump was stopped and the float removed from the well on completion of the first four pumping cycles. The well was then allowed to recharge. The recharging took about 200 days. One further pumping cycle was then carried out, the float being regulated to give a well drawdown of 2.30 ft.

Results

The usual trend of increasing drawdowns and decreasing discharge rates was observed. The drawdowns increased quickly at first and then more slowly to maximum figures. The relatively high initial discharge figure fell off quickly in the early stages, the rate of fall-off decreasing gradually. The measured steady state discharge of 11,685 cc per day was divided by 1.1955 (correction factor for partial penetration of the well) to give a corrected daily discharge rate of 9,774 cc. This figure was then used in the Dupuit formula and gave a permeability value of 0.2867 cm per day.

Discussion

The outstanding result of this experiment is the low permeability figure. The calculated value of 0.2867 cm per day is approximately the same as that calculated for the 7.03-ft well drawdown (0.2937 cm per day). This shows that even after a 200-day recharging period, the permeability did not improve and the entrapped air was not dislodged or dissolved by the rising water table. Another notable feature is the short radius of influence. This reached a maximum value of approximately 10 ft after 79 days but reduced again to about 7 ft at a later stage. The results indicate that the installation of deep drains in blanket peat can cause a permanent or semipermanent reduction of permeability. The position was not remedied in our experiment by a 7-month recharging operation. Whether or not a longer recharging period would result in increased permeability will have to be verified by long-term experiments. We can, however, say that the installation of drains at depths greater than 3½ ft offers no advantages and may result in a permanent reduction in permeability.

ACKNOWLEDGMENTS

We wish to thank the Director of the Agricultural Institute for his interest in the project and for the facilities granted in its execution; the staffs of the Soil Physics Department, the Peatland Experimental Station and the Statistics Department of the Agricultural Institute; the staff of the Soil Mechanics Department, University College, Dublin; and Michael Sheeran, Technician in charge of the field work at Glenamoy.

REFERENCES

1. Dupuit, J. Etudes Theoriques et Pratiques sur le Mouvement des Eaux (second edition). Paris, 1863.
2. Jaeger, C. Engineering Fluid Mechanics. Blackie and Son Ltd., London, 1956
3. Babbitt, H. E., and Caldwell, D. H. The Free Surface Around and Interference Between Gravity Wells. Eng. Exp. Sta., Bull. 374, Univ. of Ill., 1948.
4. Hansen, V. E. Unconfined Ground-Water Flow to Multiple Wells. Trans. Amer. Soc. Civil Eng., Vol. 118, p. 1098, 1953.
5. Boulton, N. S. The Flow Pattern Near a Gravity Well in a Uniform Water-Bearing Medium. Jour. Inst. Civil Eng., Vol. 36, p. 534, 1951.
6. Hall, H. P. An Investigation of Steady Flow Toward a Gravity Well. La Houille Blanche, p. 8, Jan. - Feb. 1955.
7. Hall, H. P. A Historical Review of Investigations of Seepage Toward Wells. Jour. Boston Soc. Civil Eng., Vol. 41, p. 251, 1954.
8. Darcy, H. Les Fontaines Publiques de la Ville de Dijon. Paris, 1856.
9. Forchheimer, P. Uber die Ergiebigkeit von Brunnen-Anlagen und Sickerschlitzen. Zeitschrift, Der Architekten-und Ingenieur-Verein, Hannover, Vol. 32, p. 539, 1886.
10. Muskat, M. The Flow of Homogeneous Fluids Through Porous Media. McGraw-Hill Book Co., Inc., New York, 1937: reprinted by J. W. Edwards, Inc., Ann Arbor, 1946.
11. Polubarinova-Kochina, P. Ya. Theory of Ground-Water Flow. Moscow, 1952 (in Russian).
12. Chapman, T. G. Two Dimensional Ground-Water Flow Through a Bank With Vertical Faces. Geotechnique, Vol. 7, p. 35, 1957.
13. Chapman, T. G. Correspondence in Geotechnique. Vol. 7, p. 142, 1957.
14. Chapman, T. G. Capillary Effects in a Two-Dimensional Ground-Water Flow System. Geotechnique, Vol. 10, p. 55, 1960.
15. Hantush, M. S. On the Validity of the Dupuit-Forchheimer Well-Discharge Formula. Jour. Geophys. Res., Vol. 67, p. 2417, 1962.
16. Harr, M. E. Groundwater and Seepage. McGraw-Hill Book Co., Inc., New York, 1962.
17. Schaffernak, F. Uber die Standsicherheit Durchlaessiger Geschuetteter Damme. Allgem. Bauzeitung, 1917.
18. Iterson, F. K., Th. van. Eenige Theoretische Beschouwingen Over Kwel. De Ingenieur, 1916 and 1917.
19. Zee, Chong-Hung., Peterson, D. F. Jr., and Bock, R. O. Flow Into a Well by Electric and Membrane Analogy. Trans. Amer. Soc. Civil Eng., Vol. 122, p. 1088, 1957.
20. Peterson, D. F. Jr. Drainage of Agricultural Lands. Edited by J. N. Luthin, Amer. Soc. of Agronomy, Madison, Wisconsin, 1957.
21. Bjerrum, L., and Huder, T. Measurement of Permeability of Compacted Clays. Proc. 4th Int. Conf. Soil Mech., London, 1957.
22. Orlob, G. T., and Radhakrishna, G. N. The Effects of Entrapped Gases on the Hydraulic Characteristics of Porous Media. Trans. Amer. Geophys. Un., Vol. 39, p. 648, 1958.
23. Galvin, L. F. Peatland Drainage Investigations. Unpub. thesis, 1965.

Accumulation of Moisture in Soil Under an Impervious Surface

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The objective of this study was to compare the long-time accumulation of moisture in a soil subgrade beneath an impervious surface with the estimated equilibrium moisture contents based on measurements of the moisture retention characteristics of the soil and the elevation of the groundwater table. Moisture accumulation due to the formation of ice was not considered.

A theoretical approach, based on thermodynamics, was used to evaluate the free energy per unit mass of water in a soil water system in terms of component free energies. Thus, the effects of adsorptive and gravitational force fields, surface tension, pressure and dissolved materials were considered.

The experimental investigation was conducted in two phases. The first phase involved the routine tasks of periodically determining soil moisture contents, soil temperatures and water table elevations under a 150-foot square impervious surface constructed of alternate layers of asphalt roofing paper and asphalt cement. The second phase was conducted to determine the soil moisture retention characteristics, in the form of desorption curves, and other properties of a series of undisturbed soil samples taken from under, and adjacent to, the impervious surface near the close of the field investigations.

EXPERIMENTAL INVESTIGATION

• THE field laboratory site was on the Iowa State University Experimental Farm at Ankeny, Iowa. The parcel selected for the investigation was on a gentle swell of undulating, glaciated land. Drainage in general was quite satisfactory. The glacial till was interspersed with lenses of sand and gravel probably associated with the ground moraine of the Des Moines lobe of the Wisconsin glacier.

Five individual test plots were selected at various positions on the 150-ft sq impervious surface. In addition to the five plots on the surface a sixth (control) plot, supporting normal vegetation, was located approximately 10 ft west of the west edge of the surface (Fig. 1). Each 10-ft sq test plot was marked off with a 1-ft grid system. The intersections of the grid lines were numbered and used as a means of control for routine weekly soil moisture sampling procedures.

The depth of the water table was determined weekly in each of 17 water table tubes and continuously in two 16-in. wells (Fig. 2). Official U.S. Weather Bureau precipitation data are given.

Soil temperatures were determined continuously under the impervious surface and under normal vegetive cover using thermocouples and a 16-point recording potentiometer. The air temperature was determined 1 ft above ground level (Figs. 3 and 4).

As a part of the second phase of this project, a series of undisturbed soil samples in Shelby tubes were taken at the field laboratory. A total of 24 holes were driven, one at each corner of each test plot, using a screw drive mechanism

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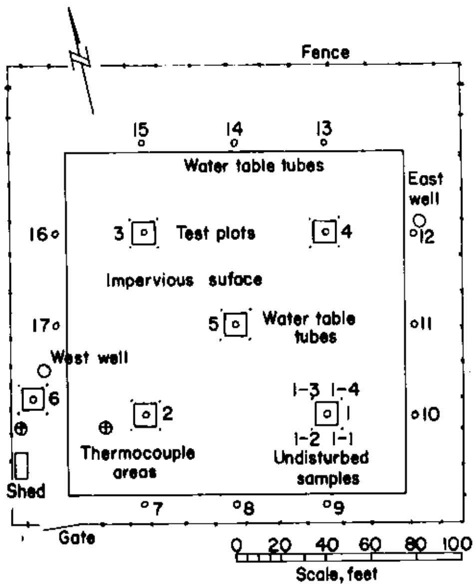


Figure 1. Planimetric layout of the field laboratory.

rather than a drop hammer. Continuous samples were taken in every case to a depth of 10 ft.

In the laboratory, soil physical characteristics were determined for each Shelby tube sample, a total of 170 in all. Desorption curves were determined using individual pressure plate apparatuses. The individual apparatus permitted the determination of an entire desorption curve using a single undisturbed soil sample that was not dislodged from the plate at any time; the entire apparatus was weighed at each pressure. Samples were approximately 2½ in. in diameter by 3 in. high and were saturated prior to the test. Other properties determined were bulk density, Atterberg limits, particle size distribution, textural classification and specific gravity. Composite desorption curves were drawn for all of the 24 test holes (Figs. 5, 6 and 7). The

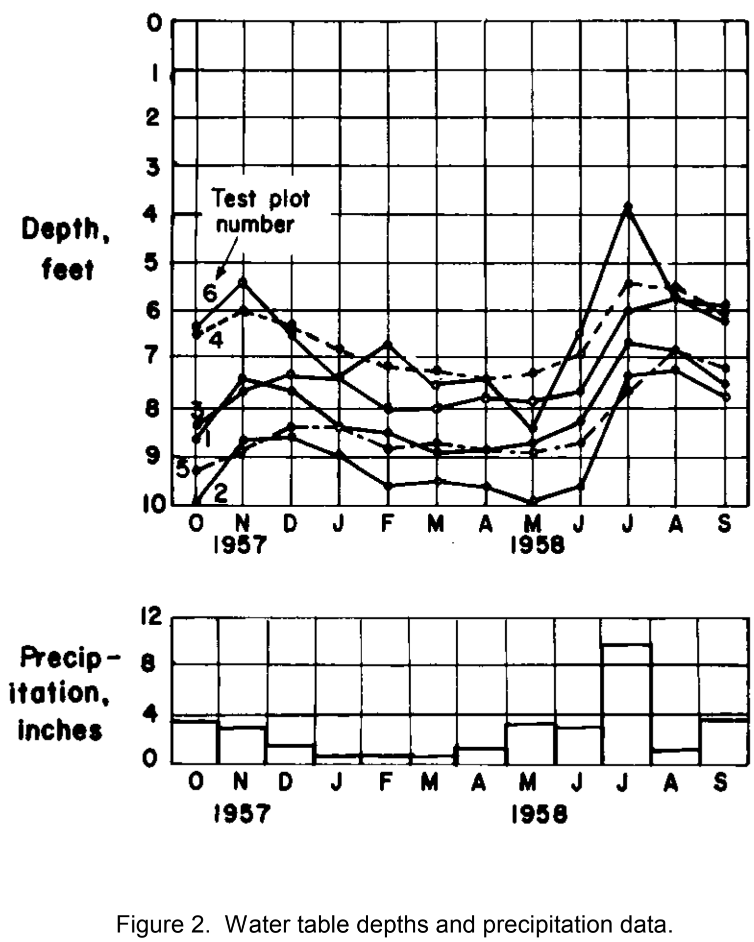


Figure 2. Water table depths and precipitation data.

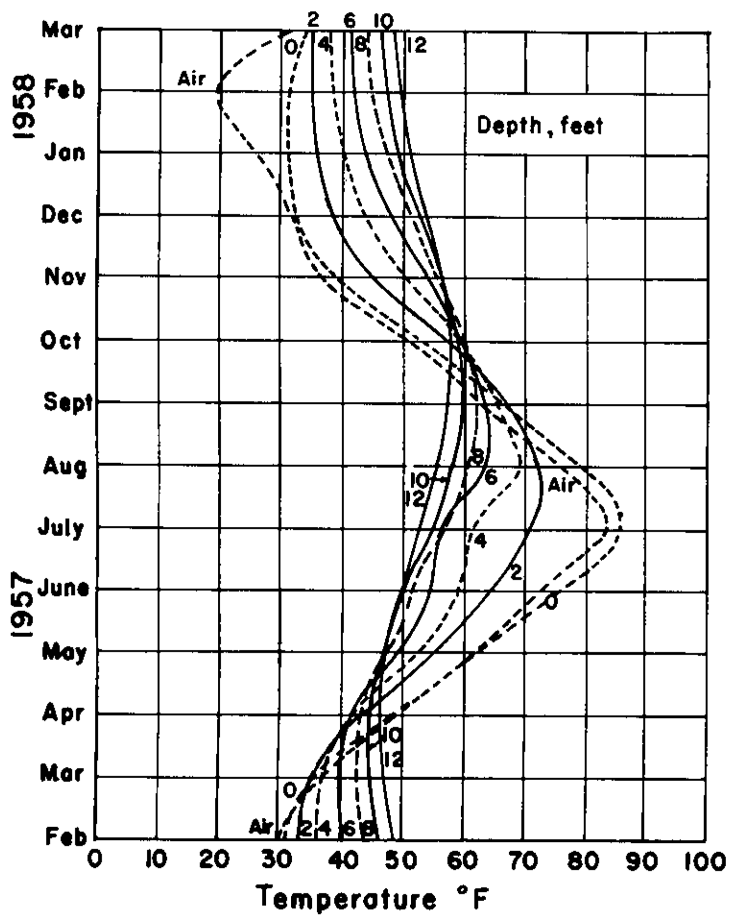


Figure 3. Soil temperatures, covered area.

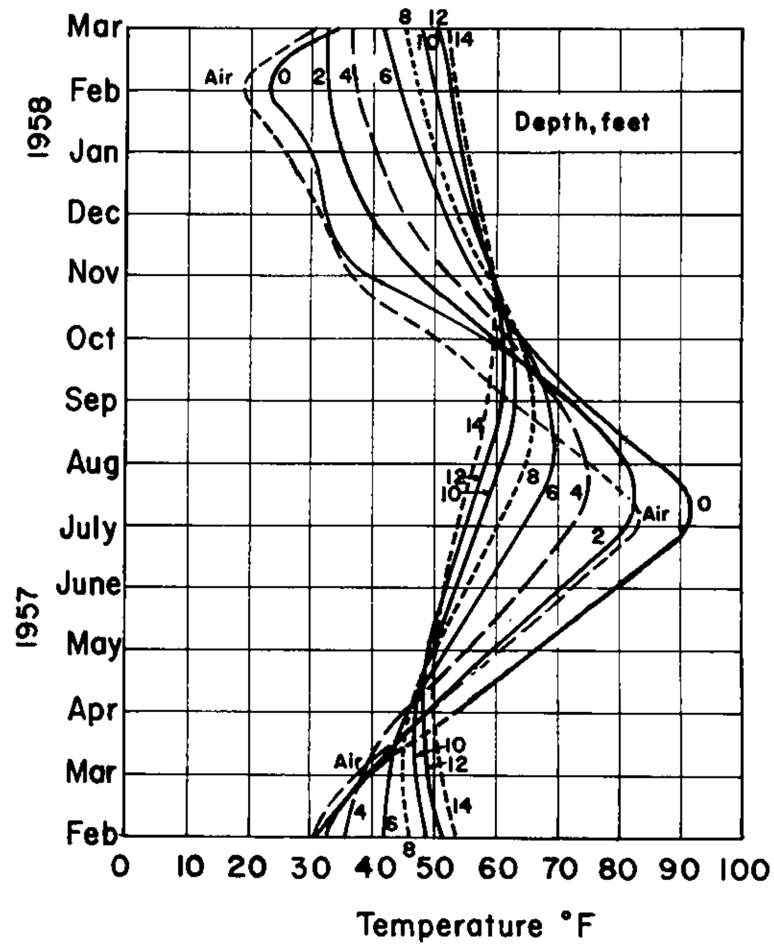


Figure 4. Soil temperatures, control area.

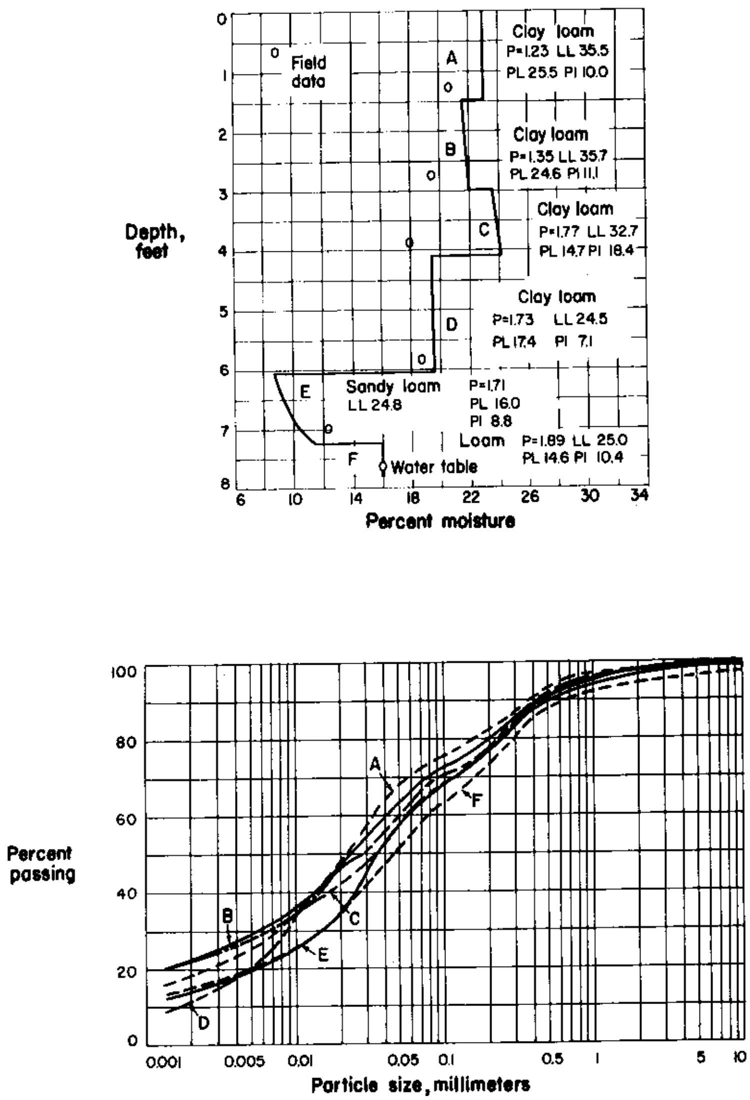
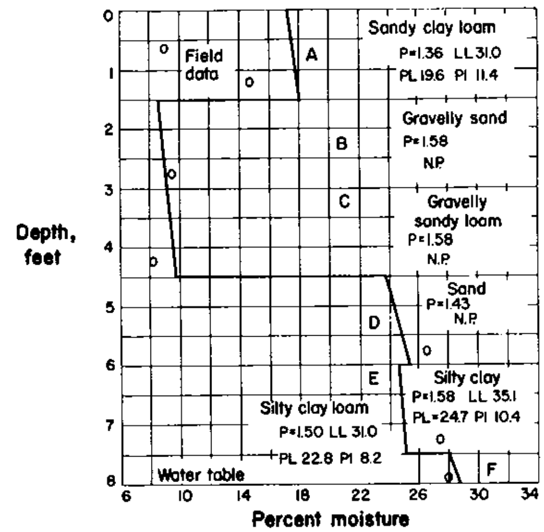


Figure 5. Test hole 1-1. Above: composite desorption curve; below: particle size distribution curves. (In Figs. 5-7 the following symbols are used: P = dry bulk density, PI = plasticity index, LL = liquid limit, NP = non-plastic, PL = plastic limit.)

field data points shown were moisture contents determined when the undisturbed samples were taken.

CORRELATION OF DATA

All soil moisture contents determined during the period October 1957 to September 1958 were tabulated (3). The tabular values were in chronological order and each test plot is listed in numerical order. Originally, direct comparisons of the field data were to be made with the appropriate desorption curves determined in the laboratory. This plan presupposed a somewhat uniform status of the soil types and environmental conditions at the test site. It was later found that such a correlation involved the simultaneous treatment of several salient variables: soil moisture content, soil characteristics, variation of soil characteristics within a given test plot, soil sample depth, water table fluctuation, time and soil temperatures. Since such a comparison was



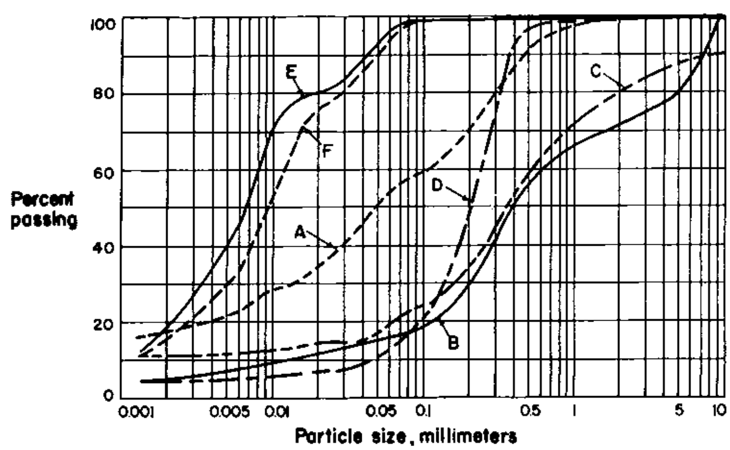


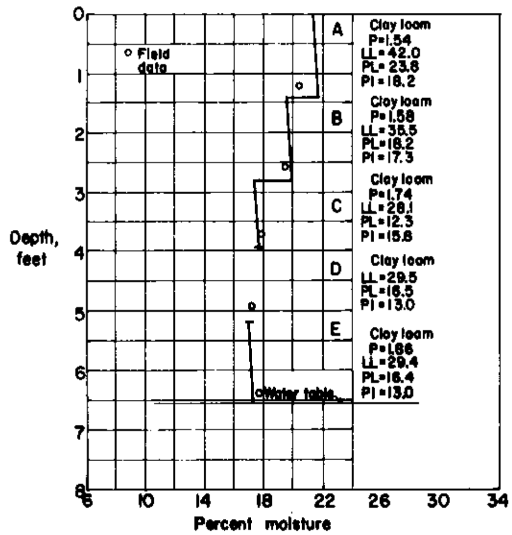
Figure 6. Test hole 2-1. Above: composite desorption curve; below: particle size distribution curves.

virtually impossible, it was necessary to make some assumptions and adjustments in the plan.

The period October 1957 to September 1958 was selected to eliminate water table fluctuation as a variable. During this period water table fluctuations were at a minimum and the individual water table tubes were in full operation.

Time was eliminated as a variable by always assuming an equilibrium condition. Obviously an equilibrium condition was never reached, but the assumption was necessary for simplication.

As a first trial the moisture contents of the undisturbed samples determined at sampling were compared with the desorption curves determined from these same samples. In so doing, a direct comparison was possible because the effect of changing soil characteristics within the test plots was eliminated and because during the sampling period (October 1958) the soil temperatures at all depths were approximately the same. The essentially constant temperatures throughout the soil profile were of the order of 50 F. This phenomenon of constant temperatures at all depths occurs semiannually as a cyclic temperature "turnover" (Figs. 3 and 4). Since the desorption



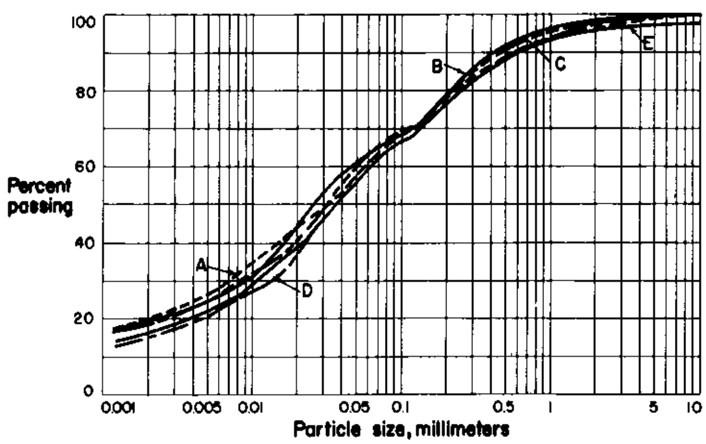


Figure 7. Test hole 6-2. Above: composite desorption curve; below: particle size distribution curve.

curves were determined at a constant temperature of 77 F, the change in moisture content caused by the different temperatures in the field and in the laboratory is probably small. The data are compared with the individual desorption curves (Figs. 5, 6 and 7), and a good correlation exists in nearly every case.

The individual soil moisture contents were too voluminous to use effectively, so it was necessary to determine average monthly moisture contents for each foot of depth for each test plot (Figs. 8 and 9). In most cases, the monthly averages represent four to five weekly moisture contents, although there were fewer determinations in some of the colder months. The moisture content of the upper 2 ft of soil in every test plot fluctuated to a considerable extent throughout the year, but all six test plots exhibited the same trend. This trend showed increasing moisture contents from October 1957 through the colder months of the period and decreasing moisture contents as the warmer months approached. Except for frost on the underside of the impervious surface, no obvious accumulation of ice was encountered. The increasing moisture contents in the upper reaches of the profiles observed during the colder months took place at a time when the water table was falling and the decreasing moisture contents

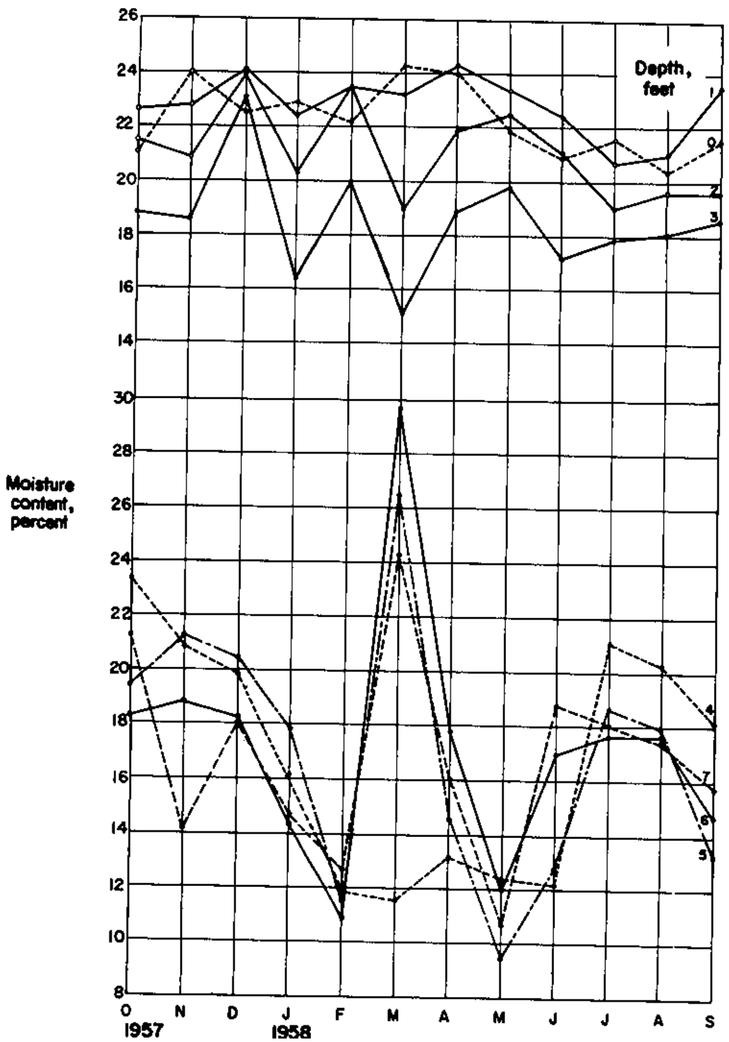


Figure 8. Soil moisture contents, test plot 5.

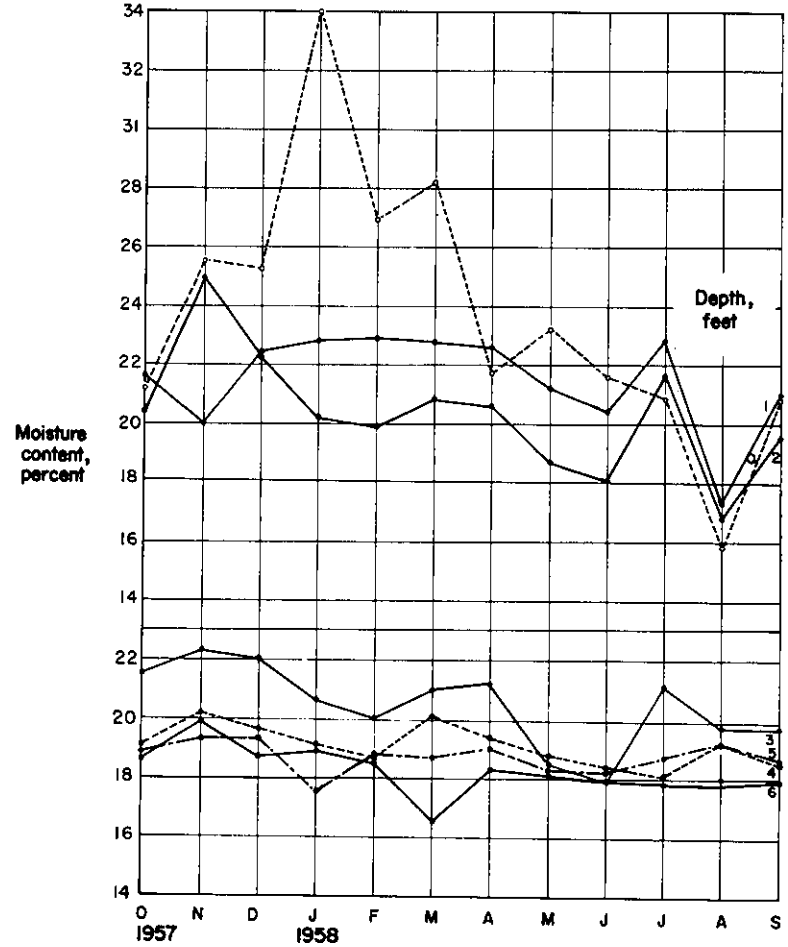


Figure 9. Soil moisture contents, test plot 6.

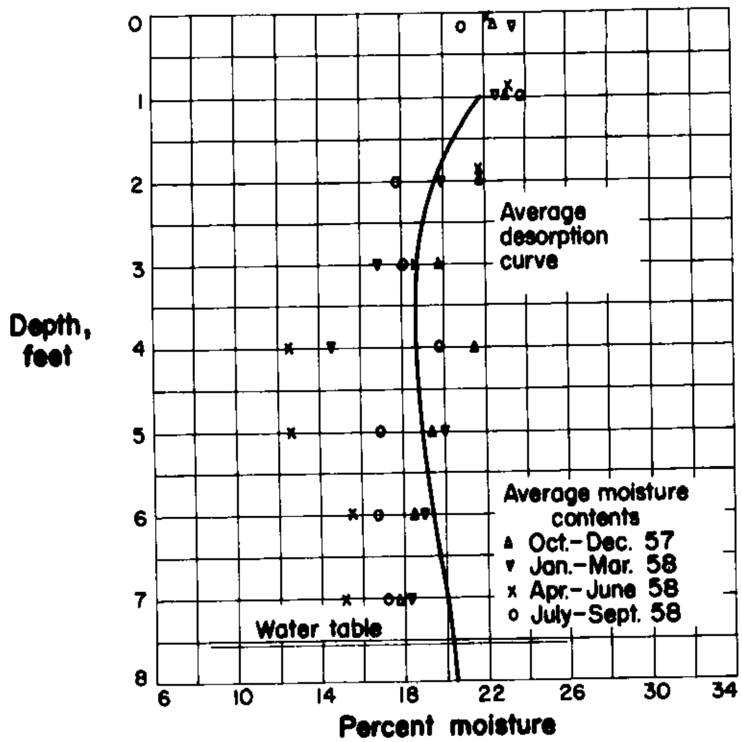


Figure 10. Master desorption curve, test plot 5.

occurring during the warmer months took place at a time when the water table was rising. Most probably this phenomenon was primarily due to changes in soil temperatures.

It was necessary to determine a master desorption curve for each test plot so that a comparison between the field data and the desorption curves could be made, and the difficulties arising from variations of soils within the test plots could be circumvented. In some cases this was done with relative ease, in others with an almost certain loss of accuracy. The four desorption curves of each test plot were given equal weight and averaged. This was done by averaging the moisture contents indicated by the four curves at various depths and then passing a smooth curve through the values obtained. The depths were chosen so they coincided with the depths from which the actual desorption samples were taken.

The weekly moisture contents were averaged for each test plot in three-month periods. These four periods are October to December 1957; January to March 1958; April to June 1958; and July to September 1958. The averages determined for these periods are compared with the six average or master desorption curves (Figs. 10 and 11). By using this system of comparison, the soil moisture contents are expressed in terms of the variables: soil sample depth as expressed as the ordinate, soil characteristics as represented by the sinuosities of the desorption curves, and temperatures as indirectly represented by the four curves determined at different times of the year. The variable resulting from the changing soil characteristics within the individual test plots was accounted for by the averaging process.

APPROXIMATE METHOD FOR PREDICTING MOISTURE CONTENTS FOR DESIGN

The formula for the height of rise in a perfectly wetted capillary may be restated as follows:

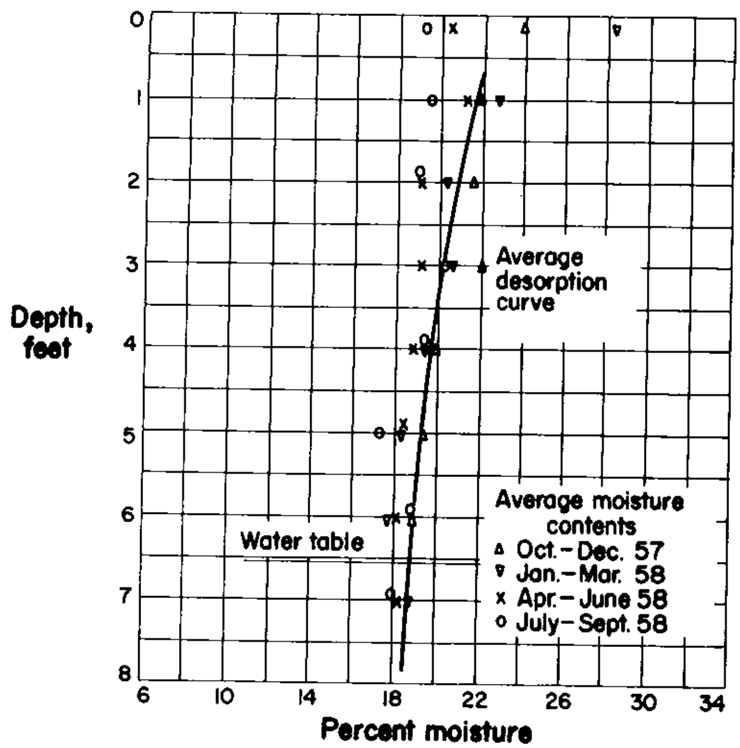


Figure 11. Master desorption curve, test plot 6.

$$\frac{r}{2} = \frac{\sigma v}{gh} \tag{1}$$

where

r = radius of curvature,
σ = surface tension of the water,
v = specific volume of the water,
g = gravitational constant, and
h = height of rise.

The radius of curvature of the menisci at a given position above the datum is not a function of the soil, although the condition of the soil greatly affects the moisture content at a given radius of curvature. Eq. 1 is idealized insofar as the radii of curvature of the menisci are stated in terms of a single radius, r. To generalize Eq. 1, the term r/2 is replaced by a parameter, r_e, which will be referred to as the equivalent radius of curvature:

$$r_e = \frac{\sigma v}{gh} \tag{2}$$

It is now possible to make a plot of the equivalent radius of curvature vs height. There will be a series of such plots, each representing a different temperature.

Now using a desorption curve of a soil under study, it is possible, by making use of a plot of the equivalent radius of curvature vs height calculated for the same temperature which was used to determine the desorption curve, to determine the equivalent radius of curvature for each moisture content of the soil. If the soil is uniform, a statement of the equivalent radius of curvature will then, under equilibrium conditions, indicate the moisture content of the soil.

A change in temperature will change the equivalent radius of curvature at a given height above the datum; the moisture content will then change so that the moisture content is in agreement with the new value of the equivalent radius of curvature. It is therefore possible to predict changes in moisture content which will occur as a result of a temperature change. Note that equilibrium moisture conditions must prevail whenever moisture contents are determined.

The method for predicting moisture contents as given is referred to as an approximate method because: first, the equivalent radius of curvature is an idealized parameter; and second, equilibrium, as such, probably never will be established.

CONCLUSIONS

At the outset of this investigation a preliminary survey was made to determine the logical site for constructing the impervious surface. Many possible sites were rejected because of gravel deposits, poor drainage or for other reasons. The selected site, as it turned out, had some advantages and disadvantages not foreseen; specifically, there existed a wealth of soil types in a small area and a wide range of soil densities were encountered.

The stratified materials encountered were an advantage because their effect on the desorption curves could be studied. Unfortunately, since the stratified materials were not uniform, additional problems were encountered in correlating the data. Occasional marked offsets were observed in the composite desorption curves; many of these were caused by changes in soil types. It is noted that a soil with a very high moisture content may be in equilibrium with an adjacent soil type with a very low moisture content. This is, of course, caused by the differences in the physical and chemical makeup of the soils. In the moisture tension range investigated, the physical characteristics of the soil probably have more effect on the moisture contents than do the chemical characteristics. The data support the conclusion that within a soil column the equilibrium moisture content of a given soil at a given moisture tension, as predicted from its sorption curve, is unaffected by stratification within the soil column (6).

It was not realized, at first, that the equilibrium moisture content of a given soil at a given moisture tension was so greatly affected by its dry density. For this reason, the soil chosen to be covered by the impervious surface was not comparable with the soil that would normally be found under a highway pavement; the density of the soil under a pavement would be greater, no doubt, and more uniform. Actually the changes in density, although introducing additional problems in correlation, were advantageous because their effect on the desorption curves was enlightening. Of particular interest is the apparently reversed trend of the composite desorption curves. For example, where there were no changes in soil type the moisture content increased with increasing height above the water table (Fig. 7). This trend is supported both by the composite desorption curves determined in the laboratory and by soil moisture contents measured in the field. Although other factors may contribute, the explanation for this behavior seems to lie in the changing soil densities. The particle size distribution curves (Fig. 7) do not indicate any appreciable differences in the mechanical analyses of the various components of the soil column. It seems, therefore, that the changing densities are caused merely by greater compaction. Apparently increased compaction changes the pore structure so that, over the range of moisture tensions investigated, the more dense form of a given soil is incapable of holding as much water at a given moisture tension as a less dense form of the same soil. Although the example cited is a special case, the above phenomenon occurs in most of the composite curves to a greater or lesser extent.

The temperature of the soil mass has only a relatively small effect on the equilibrium moisture content. This statement applies only to those ranges of soil moisture tension and temperatures investigated in this project, but the information gathered does support data presented by others (7). This observation does not include the moisture concentrations due to frost action, but only the accumulation due to the temperature differential itself.

The average moisture content at zero depth under the impervious surface in each of the test plots for the period January to March was found to be about 4.5 percent higher than the corresponding average moisture content for the period July to September (Fig. 10). Specifically, test plot number one had an average cold weather moisture content at 21.5 percent and a warm weather moisture content of 17 percent, both at zero depth. Using the proposed approximate method for estimating the change in moisture with temperature, it is found that the method estimates a change of 15 percent or a reduction of 3.3 percent moisture content from the cold period to the warm period. The 4.5 percent figure compares favorably with the 3.3 percent figure when it is considered that frost accumulation during the winter is ignored and that the average temperatures at zero depth do not reflect the true picture of the extremes; temperatures directly beneath the impervious surface were measured in excess of 120 F. Such a high temperature probably would not be possible under a pavement slab because of the thickness of the pavement as opposed to the very thin impervious layer employed in this project.

The findings of this investigation may be summarized as follows:

1. The equilibrium moisture contents in a soil column under an impervious surface can be predicted from desorption curves run on undisturbed samples of the soils providing that both the temperature and water table elevation are known.
2. Temperature has only a minor effect on the ultimate moisture contents predicted, except under extreme temperature conditions. The temperatures measured directly beneath the impervious surface during this investigation were considered to be abnormally high during the summer months and therefore rather large changes in moisture content resulted.
3. For soils such as were encountered in this investigation, the changes in moisture content attributable to changes in temperature can be predicted within close limits with the approximate method herein proposed.
4. Terminal moisture contents at various depths under an impervious surface as predicted by appropriate desorption curves are not affected by soil stratification.
5. At relatively low moisture tension values soil density has a decided effect on equilibrium moisture contents, higher moisture contents being observed at lower soil densities.
6. Under normal field conditions, where increasing soil density is noted with increasing depth, it is possible to note increasing moisture contents with increasing height above the water table.

By using the results of this study, an engineer could predict the terminal soil moisture contents under an existing or planned impervious surface. To predict the terminal moisture contents, the engineer would have to determine the desorption curves of the soils in the condition in which they occur in the embankment. In a highway pavement structure the soil samples would be compacted to the design density. The engineer would also have to predict the highest level of the water table under the surface and estimate the probable soil temperature. The proposed approximate method estimates equilibrium moisture content changes resulting from temperature differentials. It must be emphasized, however, that this method will not account for moisture accumulation due to "ice lenses," nor would it necessarily be accurate if saline soils were encountered. With this knowledge, the engineer could then determine the bearing capacity of the soil at the predicted moisture content rather than at saturation.

REFERENCES

1. Edlefsen, N. E., and Anderson, Alfred B. C. Thermodynamics of Soil Moisture. Hilgardia, Vol. 15, pp. 31-298, 1943.
2. Aitchison, G. D. (Editor). Moisture Equilibria and Moisture Changes in Soils; A Symposium in Print. Butterworths, Australia, 1965.
3. Mickle, J. L. Accumulation of Moisture in Soil Under an Impervious Surface. Unpublished PhD dissertation. Iowa State University Library, Ames, Iowa, 1960.

4. Spangler, M. G., and Mickle, J. L. Accumulation of Moisture in Soil Under an Impervious Surface. Final Report, Project 309-S, Iowa State Engineering Research Institute, Iowa State Highway Research Board, Ames, Iowa, 1961.
5. Abstract of Reference 4. In: Methods of Protecting Roads and Their Component Parts Against the Effects of Water. Final report on the survey of research in progress or recently terminated in member countries. Organization for Economic Cooperation and Development, Parts, 1965.
6. Spangler, M. G., and Pien, W. T. Distribution of Capillary Moisture at Equilibrium in Stratified Soil. HRB Spec. Rept. 2, pp. 120-125, 1952.
7. Richards, L. A., and Weaver, L. R. Moisture Retention of Some Irrigated Soils as Related to Soil-Moisture Retention. Jour. of Agr. Research, Vol. 69, pp. 215-235, 1944.

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Supported by private and public contributions, grants, and contracts, and voluntary contributions of time and effort by several thousand of the nation's leading scientists and engineers, the Academies and their Research Council thus work to serve the national interest, to foster the sound development of science and engineering, and to promote their effective application for the benefit of society.

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The HIGHWAY RESEARCH BOARD, an agency of the Division of Engineering, was established November 11, 1920, as a cooperative organization of the highway technologists of America operating under the auspices of the National Research Council and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the Board are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.