HIGHWAY SUBDRAINAGE

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SYNOPSIS

While the importance of subdrainage has long been recognized, highway drainage designs have generally been based on empirical rules. Although this report does not attempt to establish design criteria, it presents test methods and data on permeability and drainability of soil and indicates their application to highway subdrainage.

Results of permeability tests on various materials with appropriate apparatus show the importance of detailed specifications of procedure and materials and the extremely wide range of possible values. For instance, the permeability of a material depends on the method of its compaction as well as its density, gradation and plasticity.

Substitution of test results in appropriate formulas illustrates the effect of boundary conditions and the coefficient of permeability on the rate of drainage. It is shown that the small gradients available for lateral drainage of base courses prevent rapid drainage of dense-graded materials on impervious subgrades. Even after drainage a dense-graded material will hold considerable water by capillarity if protected from evaporation. While open-graded materials will drain more readily, provision must be made to prevent intrusion of fine subgrade soil, and it is difficult to compact them so that traffic will not cause further displacement.

Since both density and drainability are desirable, the range of satisfactory materials is limited, and it is sometimes necessary to choose which property should be given preference.

● John McAdam, in 1824, wrote that water with alternate freeze and thaw are the evils to be guarded against and, after having secured the soil from under water, the roadmaker should then secure it from rainwater. The paramount importance of drainage with respect to stability of roads is still recognized. However, there is wide difference of opinion as to what constitutes good drainage and how it is to be effected.

This report does not presume to fix design criteria but rather to present some test data on permeability and drainability of soil and to indicate their application to highway subdrainage. Various laboratory and field methods of determining permeability are reported for soils representing the classification groups, for several bituminous paving mixtures, for sieve fractions of sand, concrete sands, and clean aggregates with different minimum sizes, and for graded sand gravels and sands with various amounts and types of material passing the No. 200 sieve. Water held by these latter mixtures after drainage is also reported. The data are used to illustrate methods of calculating vertical capillary flow, flow into horizontal drains, and lateral drainage of base courses. This is followed by a discussion of the interrelation of drainage, density, and gradation of soils as they affect the problems of bearing capacity, intrusion, and pumping.

PERMEABILITY TEST METHODS

For small velocities the rate of flow of water through soil is given by the equation:

$$Q = kAh/d \qquad (1)$$

where

- Q = volume of flow per unit time.
- k = coeficient of permeability.
- A = gross area of soil perpendicular to direction of flow.
- h = head loss in a distance d through the soil in the direction of flow.

If a constant head is maintained on a soil sample in a laboratory test, equation 1 may be used to solve for k, thus:

$$k = Qd/Ah \qquad (2)$$

For two layers in series, such as the sample and its pervious support, with thicknesses d_1 and d_2 and permeabilities k_1 and k_2, respectively, the over-all value of k is $(d_1 + d_2)/(d_1/k_1 + d_2/k_2)$ or, in general, $\Sigma d/\Sigma(d/k)$. If k_2 is large enough, the effect of the support is negligible. For instance, if $k_2 = 100\ k_1(d_2/d_1)$, neglect of the support introduces an error of only 1 per cent.

In order to facilitate measurement of a small volume Q, a falling head is often used, as illustrated in Figure 1. The equation for calculating k, shown in Figure 1, is obtained by integration of Equation 2, to take care of the fact that the head decreases continuously during the test, and is:

$$k = (2.3ad/At) \log (h_1/h_2) \qquad (3)$$

The test procedure is as follows: A sample trimmed to size from an undisturbed core, or the desired amount of loose material, is

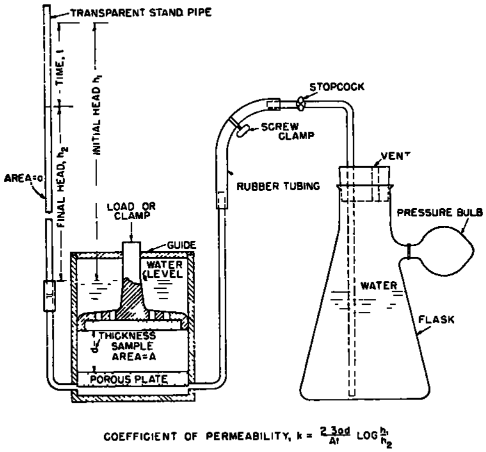


Figure 1. Apparatus for measuring permeability by means of a falling head.

placed in the apparatus. The piston is placed on the soil and loaded or clamped at a given sample thickness. The sample is inundated. The pressure bulb is used to force water from the flask to fill the standpipe from below and thus prevent air from being trapped below the sample. The stopcock is closed when the water is approximately at the initial head. For the less pervious materials, the final adjustment in the initial head is made with the screw clamp. The time required for the water to fall to the final head is recorded.

In a typical example, a = 0.09 square centimeter, A = 81.08 square centimeters, d = 1.5 inches, h_1 = 80 centimeters, and h_2 = 40 centimeters. Then k, from Equation 3, = $0.00115/t$ in. per min., or $0.138/t$ ft. per day. The dimension "feet per day" is not the velocity of flow through the soil but a contraction of cubic feet of water per day per square foot of soil for a hydraulic gradient h/d of unity.

The magnitude of the coefficient of permeability may be judged by comparing it with the rate at which water will percolate vertically into a wet soil with a deep water table. For this condition, the hydraulic gradient is unity and the coefficient of permeability is equivalent to the rate of rainfall which could be taken into the soil if the water were uniformly distributed over the surface of the soil. Thus, a soil with k = 1 foot per day could transmit vertically downward a

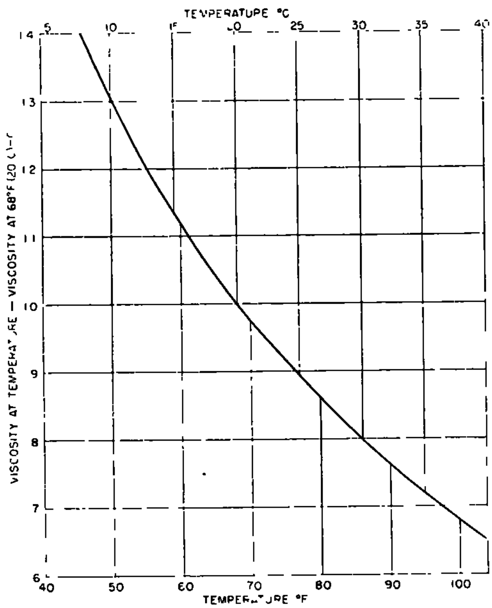


Figure 2. Effect of temperature of water on viscosity ratio.

maximum rainfall of 12 inches in 24 hours. On the other hand, a soil with k = 0.001 foot per day would require nearly 3 years to transmit 12 in. of water.

Since the permeability depends upon the viscosity of the water, which is a function of temperature, the calculated permeability at an abitrary temperature (68F. = 20C.) is often reported as $k_{68} = Ck$, where C is the viscosity of water at the test temperature divided by its viscosity at 68F. Values of C are plotted in Figure 2. Actually, the temperature also may affect properties of the soil as well as properties of the water so that it is best to make the test at the temperature of the soil in place.

The Soil Water

The soil water should be used as the peremeating liquid since the mineral and gas content of the water is difficult to duplicate. For instance, de-aired water, from which soluble gases have been removed, has sometimes been used for permeability tests. This has been done to prevent air from collecting in the soil and causing reduction in permeability, although this reduction may actually occur under certain field conditions. Similarly, water with either greater or less salt concentration than the soil water may markedly change the permeability of the soil, so that use of distilled water as a standard is not always desirable. The test results reported hereafter were obtained with clean tap water.

The use of the soil water is often impracticable. However, the permeability of saturated fine-grained soils may be determined by laboratory consolidation of undisturbed samples. If the rate at which the thickness of a sample changes after the application of a load increment is observed, the permeability may be computed from an equation based on the theory of consolidation:

$$k = 132 \frac{d\Delta d}{t_{90}\Delta p} \text{ ft. per day} \qquad (4)$$

where

d = initial specimen thickness, in inches.

Δd = reduction in thickness for a load increment Δp, in pounds per square foot.

t_{90} = factor derived from time-thickness relations, in minutes.

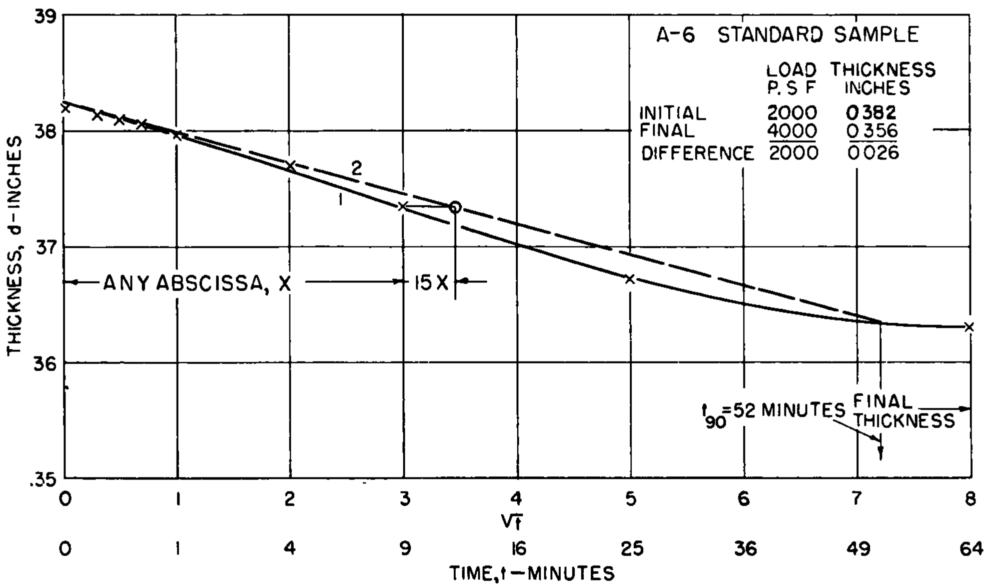


Figure 3. Thickness change with time.

To calculate t_{90}, the thickness is plotted against time as in Figure 3. The initial portion of the test relation, Line 1, is approximately a straight line. Through the intersection of Line 1 with the vertical axis, Line 2 is drawn with abscissas 0.15 greater than Line 1. Line 2 intersects the test relation at t_{90}. Thus, the square root of t_{90} = 7.2 gives t_{90} = 52. Substituting in equation 4, and using the values shown in Figure 3:

$$k = 132 \frac{0.382 \times 0.026}{52 \times 2{,}000}$$

$$= 0.000013 \text{ ft. per day.}$$

This affords an excellent test method for homogeneous materials. Since this test depends upon the rate at which water is squeezed from the soil, the results are not affected to much extent by local conditions, such as root holes, which may be the controlling factor in flow directly through the soil as in the usual permeability test. For two parallel conductors with cross-sectional areas a_1 and a_2 and permeabilities k_1, and k_2, the over-all permeability is $k = (k_1a_1 + k_2a_2)/(a_1 + a_2)$ or, in general, $k = \Sigma ka/\Sigma a$, so that the more pervious conductor has a dominant influence on the over-all value. Comparison of this relation with that for conductors in series shows that, for stratified deposits, the permeability in the direction of stratification is always greater than that perpendicular to the layers.

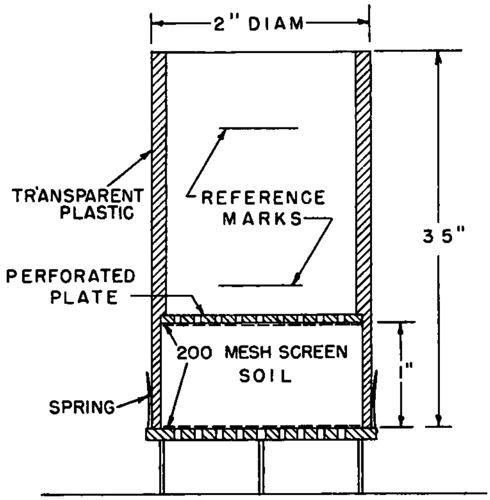


Figure 4. Permeameter for granular soils.

Because of the limited permeability of the porous plates in the apparatus shown in Figure 1, it is not used for granular soils. Figure 4 shows a device used for sands wherein the sample is retained by 200-mesh screen wire. The same falling-head principle is used but because of the higher permeability the standpipe has a larger area. For field work, a sample may be taken by forcing the device with the base removed into the soil, inverting, and striking off the excess.

To prevent turbulence and minimize migration of particles in testing coarse sands and gravels or base-course mixtures under small gradients, the device shown in Figure

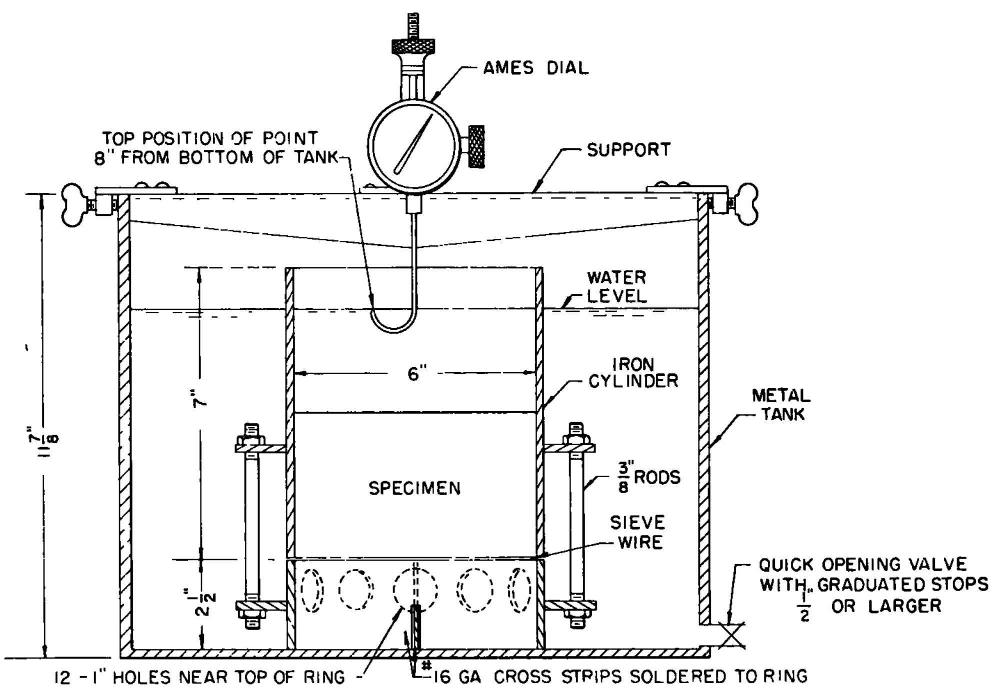


Figure 5. Drainage-lag permeameter.

5 was developed. The sample may be compacted to any desired density by either an impact or static method. The cylinder containing the compacted sample is inundated in water in the tank. The water level is allowed to come to equilibrium and its level is determined with the hook gage, which is then lowered an arbitrarily selected amount h. The valve at the bottom of the tank is opened and the outflow caught while a stopwatch records the time until the inner water level reaches the hook, at which time the watch is stopped and the valve closed.

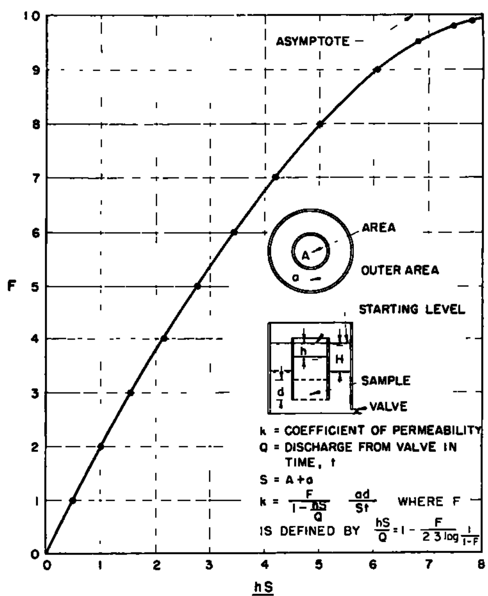


Figure 6. Permeability from drainage-lag device.

The effective head is a variable (H-h) where H is the water level lowering outside of the sample. The time, outflow, and lowering of water inside the tube holding the specimen are used in Figure 6 with the fixed dimensions of the specimen and apparatus to determine the coefficient of permeability. The formula for k in Figure 6 was derived from Equation 1 by integration, assuming a constant rate of discharge.

When it is desired to saturate samples under a vacuum before testing, the apparatus shown in Figure 7 is used. It may also be used in place of the apparatus shown in Figure 1 for materials with relatively high permeability. Water enters at C and air is removed from below the specimen by a tube attached at F to the bottom of the perforated plate which supports a piece of 200-mesh screen wire. When the water reaches the specimen, tube B is closed and water is allowed to pass through and to a height approximately one-half inch above the specimen. C is then closed and water is entered at A until the standpipe is full. The test is performed by allowing water to run out at D or E.

Field Permeability Tests

As shown in Figure 8, the coefficient of permeability may be determined on soil

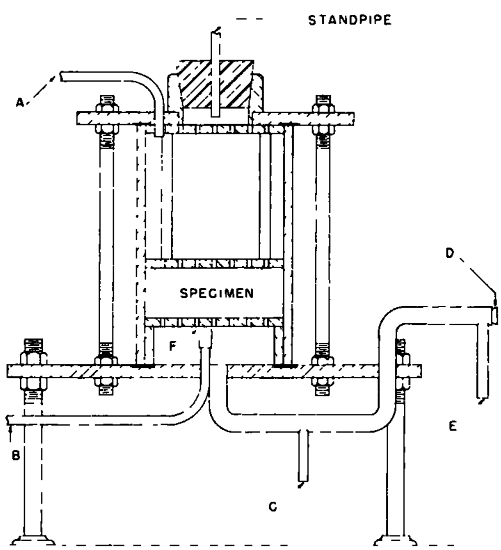


Figure 7. Permeameter.

in place below the water table by drilling a hole in the soil and measuring the water which flows from the soil into the hole (*1*, *2*).

The calculated coefficient is an average permeability for soil near the hole, if the soil is not definitely stratified or fissured. The formulas may also be used for water flowing into the soil but there is danger of error in this application because of clogging of the soil surface with suspended particles. If the flow is into unsaturated soil, allowance must be made for capillary forces.

PERMEABILITY TEST RESULTS

Coefficients of permeability for soils of variable grain size and plasticity are shown

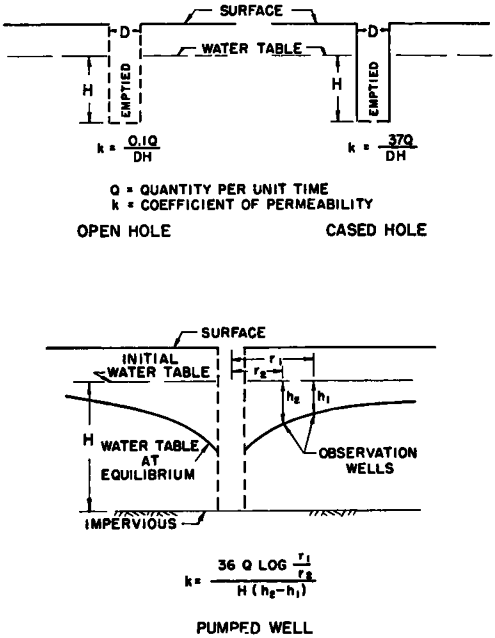


Figure 8. Determination of coefficient of permeability in field below water table.

in Table 1. The soil classification is that published by the Bureau of Public Roads in 1942. All of the tests were made on the soil fraction passing the No. 10 sieve. The device shown in Figure 1 was used for testing all soils except the A-3 sample, for which the device shown in Figure 4 was used. With the exception of the A-3 soil the results shown under the heading "compacted wet" were obtained with test samples prepared by wetting the soil to the liquid limit and compacting it in the cylinder of the device shown in Figure 1 under static loads of 1, 2, and 4 kips per square foot. The A-3 soil, a cohesionless sand, was dampened and placed in the cylinder by pressing thin layers firmly into place with a spatula. The test samples for the "compacted dry" condition were molded from air-dry soils under static loads of 1, 2, and 4 kips per square foot, inundated, allowed to drain, and the permeability determined.

The data in table 1 show that for each soil the permeability coefficient (k value) decreased with increase in density, and

TABLE 1
PERMEABILITY OF SOILS

B.P.R. group	Liquid limit	Plasticity index	Amount passing No. 200 sieve	Consolidating load	Compacted wet		Compacted dry	
					Permeability coefficient	Dry density	Permeability coefficient	Dry density
	Percent	*Percent*	*Percent*	*Kips/sq.ft.*	*Ft./day*	*Pcf.*	*Ft./day*	*Pcf.*
A-1	23	8	26	1	0.00045	114	0.0048	117
				2	.00036	115	.0020	118
				4	.00024	117	.0017	118
A-2	28	11	40	1	.00048	103	.0047	99
				2	.00028	106	.0016	102
				4	.00018	109	.0009	108
A-3	NP	NP	0	[a]	200	106	226	101
A-4	33	12	99	1	.00030	94	.00075	90
				2		97	.00060	94
				4	.00022	100	.00024	99
A-5	35	6	35	1	.021	84	.37	78
				2	.015	86	.25	80
				4	.012	90	.17	84
A-6	72	45	86	1	.000035	75	.00011	89
				2	.000019[b]	80	00011	90
				4	.000009[b]	86	00010	92
A-7	67	34	71	1	.00051	77	.026	78
				2	.00023	81	.0084	81
				4	.00020	86	0012	83
A-8	78	8	38	1	.0039	43	.041	47
				2	.0010	46	.018	48
				4	.00046	49	.010	50

[a] Patted with spatula.
[b] Derived from thickness change with time. Other values from falling-head permeability test.

that there is a wide range in the permeabilities of soils of the different groups tested. Also, soils having the same density may have widely different k values. For example, test samples in the A-5, A-6, and A-7 groups each having densities of 86 pounds per cubic foot compacted wet had permeability coefficients of 0.015, 0.000009, and 0.0002 foot per day, respectively. Also, while the dry density of the A-3 cohesionless sand and of the A-2 soil compacted wet under 2 kips per sq. ft was 106 lb. per cu. ft, the permeability coefficient for the sand was approximately 700,000 times greater than that of the A-2 soil.

The relatively high permeability and low density of the micaceous A-5 soil is probably due to the plate-like mica particles that cannot be compacted into a dense structure. The differences in the permeabilities of the same soils compacted by the two different methods described are significant. The results in table 1 show that the samples molded from air-dry materials had k values up to 50 times greater than those molded from soils wetted to the liquid limit. These higher values may be attributed to the differences in structure caused by the methods of molding. The soils compacted dry have a less uniform particle arrangement which results in the higher permeabilities.

Permeability of Pavements

Since pavements are placed over soils, their relative permeability is of interest. The permeability of homogeneous portland cement concrete without cracks or honeycombed structure (*3*) is of the order of magnitude of the permeability of the clay samples A-6 and A-7 in Table 1.

The coefficients of permeability for various samples of bituminous concrete compacted in the laboratory under a static load of 3,000 psi. and then heated to 140F. for 24 hr. were determined as shown in Table 2. The coefficient of permeability determined for one sample of bituminous concrete taken from a highway was found to be 0.00002 ft. per day. The low permeability found in the field is probably due to traffic compaction, particularly near the surface.

Permeability of Sands

The permeabilities of sieve fractions of Potomac River sand, determined in an apparatus similar to that shown in Figure 4 are given in Table 3. (The capillary height will be discussed later). Based on published tests of sand under various gradients (*4*), the approximate maximum hydraulic gradient for which Equation 1 is applicable is shown in the last column of Table 3. Equation 1 is for streamlined or laminar flow. For higher gradients turbulence reduces the flow considerably, so that this gradient becomes a limiting value for testing unless turbulent flow is to be encountered in the field. Ordinarily, for saturated flow, a gradient of one, corresponding to vertical infiltration into a wet soil without ponding, is the maximum encountered in the field. Thus, appreciable turbulence would be found only with materials whose permeability is greater than about 500 ft per day.

TABLE 2
PERMEABILITY OF BITUMINOUS CONCRETES

Percentage passing No. 200 sieve	Percentage of asphalt cement	Percentage of air voids	Permeability coefficient
Percent	*Percent*	*Percent*	*Ft./day*
5	6	11	0 45
5	6	9.9	.84
7	5.5	9.8	.39
7	6	5.6	.16

TABLE 3
PERMEABILITY OF SIEVE FRACTIONS OF SAND

Sand fraction		Saturated capillary height	Permeability coefficient k	Average grain size D	Gradient causing turbulence $300/kD$
Passing sieve No.	Retained on sieve No.				
		In.	*Ft./day*	*Mm.*	
10	20	2.5	1,430	1 183	0.2
20	30	3.7	665	.693	.6
30	40	5.2	380	.491	2
40	60	7.9	190	.313	5
60	80	11.7	160	.207	9
80	100	14.0	75	.162	25
100	140	18.5	45	.123	50
140	200	26.4	20	.087	200
200	270	35.6	9	.062	500

Table 4 shows the gradations and permeabilities of three concrete sands whose gradings represent the range of sizes usually permitted in specifications. Their permeabilities correspond approximately to those given in Table 3 for the same effective size—that size than which 10 percent by weight is smaller. For instance, with an effective size equal to the No. 80 sieve, the medium sand has a permeability of 113 compared to 118 from Table 3 (average for Nos. 60–80 and Nos. 80–100 sieve fractions). While the effective size is useful for sands with a small size range, it is not applicable to materials with a large size range such as the gravels whose permeabilities are given in Table 5.

TABLE 4
PERMEABILITY OF CONCRETE SANDS

	Fine sand	Medium sand	Coarse sand
Percentage passing:			
No. 4 sieve	100	100	100
No. 10 sieve	100	82	62
No. 20 sieve	67	49	35
No. 40 sieve	42	28	17
No. 60 sieve	25	15	8
No. 140 sieve	4	0	0
No. 200 sieve	0	0	0
Coefficient of permeability, ft. per day	63	113	194

TABLE 5
PERMEABILITY OF GRADED AGGREGATES

	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
Percentage passing:						
¾-inch sieve	100	100	100	100	100	100
½-inch sieve	85	84	83	81.5	79.5	75
⅜-inch sieve	77.5	76	74	72.5	69.5	63
No. 4 sieve	58.5	56	52.5	49	43.5	32
No. 8 sieve	42.5	39	34	29.5	22	5.8
No. 10 sieve	39	35	30	25	17	0
No. 20 sieve	26.5	22	15.5	9.8	0	0
No. 40 sieve	18.5	13.3	6.3	0	0	0
No. 60 sieve	13.0	7.5	0	0	0	0
No. 140 sieve	6.0	0	0	0	0	0
No. 200 sieve	0	0	0	0	0	0
Dry density, lb. per cu. ft.	121	117	115	111	104	101
Coefficient of permeability, ft. per day	10	110	320	1,000	2,600	3,000

Permeability of Graded Aggregates

The material graded from the ¾-in. sieve to the No. 200 sieve, shown as sample 1 in Table 5, was designed to represent the middle of the specification of the American Association of State Highway Officials for this fraction of base-course materials. The other gradings were obtained by omitting the fractions below various sieves. The apparatus shown in Figure 5 was used for these materials. The wide range of permeabilities is to be noted as well as the decrease in density as the fines are progressively omitted. The finer fractions have a predominant effect on the permeability, as shown by the increase from 10 to 110 in the k values obtained by omitting the 6 percent between the No. 140 and No. 200 sieves from the first mixture.

To determine the effect of material passing the No. 200 sieve on the permeability of aggregate mixtures, several types and quantities of fines were added to Potomac River sand and gravel graded between the ¾-in. and No. 200 sieves. The standard AASHO compaction test was made on each mixture except that the material retained on the No. 4 sieve was not removed.

For the permeability test, the material was mixed with water to obtain a moisture content 20 percent greater than the optimum (to obtain minimum permeability), tamped to a density slightly below maximum, loaded statically to obtain and maintain maximum density, and saturated from below with tap water. The permeability was determined on duplicate samples in a device similar to that shown in Figure 1, except that for permeabilities greater than 0.1 foot per day the apparatus shown in Figure 7 was used. After the permeability test, one sample was dried at 110C., washed on a No. 200 sieve, and the sieve analysis determined. The other sample was air-dried, the sieve analysis determined by the standard AASHO method, and the fraction passing the No. 40 sieve was tested according to standard AASHO procedures to determine the physical characteristics.

Comparison of the average gradations obtained by washing with the design gradations, for each percentage of admixture, as shown

TABLE 6
GRADATION OF PERMEABILITY SAMPLES FOR BASE-COURSE MIXTURES PASSING ¾-IN. SIEVE

Admixture and method of determination	Percentage passing sieve										
	½-inch	⅜-inch	No. 4	No. 8	No. 10	No. 20	No. 40	No. 60	No. 100	No. 140	No. 200
No admixture:											
Design	86	73	52	37	34	20	12	7	4	2	0
Washed	86	75	54	39	36	23	15	11	7	4	3
Standard		75	56		43		23				6
5-percent admixture:											
Design	86	74	55	40	37	24	17	12	9	7	5
Washed, average	86	75	55	40	38	26	18	13	10	8	6
Standard, average		75	56		41		24				9
10-percent admixture:											
Design	87	75	57	43	40	28	21	17	14	12	10
Washed, average	87	75	57	44	42	29	22	18	14	12	11
Standard, average		75	58		45		28				14
15-percent admixture:											
Design	88	77	59	46	44	32	26	21	18	16	15
Washed, average	88	77	60	46	44	33	26	22	19	17	16
Standard, average		77	60		48		32				18
25-percent admixture:											
Design	89	79	64	52	50	40	34	31	28	26	25
Washed, average	88	79	63	52	50	40	34	30	28	26	25
Standard, average		78	65		53		40				27

TABLE 7
PHYSICAL CHARACTERISTICS AND PERMEABILITY OF GRADED AGGREGATE WITH ADMIXTURES OF VARIOUS AMOUNTS AND TYPES OF MATERIAL PASSING THE NO. 200 SIEVE

Type and percentage of admixture	Liquid limit	Plasticity index	Compaction test		Molding moisture	Permeability coefficient (68F.)		
			Maximum density	Optimum moisture		Test 1	Test 2	Average
			Pcf.	*Percent*	*Percent*	*Ft./day*	*Ft./day*	*Ft./day*
None	NP	NP	134.5	8.0	9.6	12	13	12
Silica dust:								
5 percent	NP	NP	136	7.0	8.4	.61	.70	.66
10 percent	NP	NP	138	6.2	7.4	.058	.061	.060
15 percent	NP	NP	137	6.6	7.9	.018	.022	.020
25 percent	NP	NP	132	8.2	9.8	.016	.020	.018
100 percent	NP	NP	103	18.3	22.0	.025	.026	.026
Limestone dust:								
5 percent	NP	NP	137	6.8	8.2	.33	.44	.38
10 percent	NP	NP	142	5.6	6.7	.02	.06	.04
15 percent	NP	NP	141	5.6	6.7	.02	.03	.025
25 percent	NP	NP	138	7.2	8.6	.015	.018	.016
100 percent	NP	NP	99	19.6	23.5	.024	.026	.025
Manor loam:								
5 percent	NP	NP	137	7.0	8.4	.08	.11	.10
10 percent	NP	NP	137	6.0	7.2	.02	.02	.02
15 percent	25	4	136	6.5	7.8	.007	.007	.007
25 percent	29	7	132	6.7	8.0	.0026	.0029	.0028
100 percent	38	8	98	22.4	26.9	.0009	.0017	.0013
Keyport silt loam:								
5 percent	16	2	138	6.5	7.8	.043	.067	.055
10 percent	18	6	139	6.1	7.3	.0005	.0015	.0010
15 percent	19	6	140	5.5	6.6	.00009	.00010	.00010
25 percent	21	8	135	6.6	7.9	.00007	.00008	.00008
100 percent	33	14	112	17.0	20.4	.00004	.00004	.00004
Tuxedo clay:								
5 percent	19	5	138	7.0	8.4	.005	.015	.010
10 percent	23	8	137	6.2	7.4	.00030	.00039	.00034
15 percent	26	11	135	5.8	7.0	.00007	.00009	.00008
25 percent	35	17	131	8.0	9.6	.00004	.00006	.00005
100 percent	55	28	106	20.4	24.5	.00002	.00002	.00002

in Table 6, indicates some degradation due to compaction. Comparison of the gradations obtained by washing with the gradations obtained by the standard method indicates a further degradation due to pulverizing the air-dry material and the mechanical dispersion used for the hydrometer analysis.

The physical characteristics and permeabilities of the original material and the various mixtures are shown in Table 7. It is to be noted that these permeabilities are for samples packed very uniformly in the laboratory. Lack of uniformity in the field

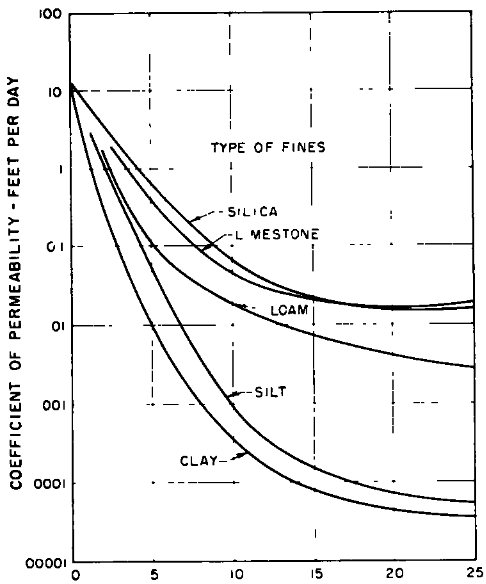


Figure 9. Effect of fines on permeability of graded aggregate.

may give areas of very high permeability or layers and dams of relatively low permeability.

The data in Table 7 show that a 5-percent admixture causes a marked decrease in permeability, even for the nonplastic admixtures, as illustrated in Figure 9. For the same percentage of admixture the permeabilities vary widely with the type of admixtures. With increasing percentages of admixture, the permeabilities approach the values obtained for the admixture alone (100-percent admixture). While the plasticity and gradation both affect the permeability, these values are not sufficient to determine the coefficient of permeability. This is to be expected since the permeability depends upon the arrangement of the particles (structure), which are not considered in the classification tests.

Similar tests were made on graded sand with admixtures of various percentages of fines using the permeameter shown in Figure 7. Samples were compacted to maximum density at optimum moisture by the standard AASHO compaction procedure.

The gradation of the mixtures as compounded is shown in Table 8, and the compaction and permeability test results are shown in Table 9. The results are comparable to those for the gravel-sand fines for the same ratio of fines to sand. Thus the 5-percent admixture in Table 7 is roughly comparable to a 9-percent admixture inter-

TABLE 8
GRADUTION OF PERMEABILITY SAMPLES FOR BASE-COURSE MIXTURES PASSING THE NO. 4 SIEVE

Admixture, percent of total weight	Percentage passing sieve						
	No. 10	No. 20	No. 40	No. 60	No. 100	No. 140	No. 200
0	71	46	30	18	10	4	0
5	73	48	33	22	14	8	5
10	74	51	36	26	19	13	10
15	75	54	40	30	23	18	15
25	78	59	47	39	32	28	25

polated in Table 9, since the sand and fines constitute only 57 percent of the gravel-sand-fines mixture. The permeability of 0.10 for 5-percent Manor loam in Table 7 is about equal to the value for 9-percent interpolated in Table 9. The permeability of the plastic gravel-sand-fines mixtures tends to be somewhat lower due to the fact that the moisture content at the time of compaction was 20 percent above optimum rather than optimum as for the sand-fines.

CAPILLARY STORAGE

In soils which have not been waterproofed to reduce their affinity for water, the surface tension of the water produces an appreciable force which can hold water in the soil above the water table.

The height above the water table to which a soil can stay saturated may be determined by means of the apparatus shown in Figure 10. Starting with the water in both tubes at the level of the top of the soil, the stopcock is opened, allowing water to drain slowly from the righthand tube until the water in this tube rises temporarily when air is first drawn through the soil specimen. The difference between this level and the bottom of the soil specimen is the saturated capillary height shown in Table 3 for several sand fractions. For fine-grained soils this test is quite sensitive to changes in density and uniformity of structure since it is a measure of the largest pore.

TABLE 9

PERMEABILITY OF GRADED SAND PASSING NO. 4 SIEVE WITH ADMIXTURES OF VARIOUS AMOUNTS AND TYPES OF MATERIAL PASSING THE NO. 200 SIEVE

Type and percentage of admixture	Molding		Permeability coefficient (68F.)		
	Density	Moist-ure	Test 1	Test 2	Aver-age
	Pcf.	*Percent*	*Ft./day*	*Ft./day*	*Ft./day*
None	119.5	11.5	4	5	4 5
Silica dust.					
5 percent.	123.0	10.0	.50	.84	.67
10 percent.	127 0	9.0	.10	.11	.10
15 percent.	126.5	8.5	.047	.050	.048
25 percent.	127.5	8.0	.017	.021	.019
Linestone dust:					
5 percent..	123.0	7 0	.44	.59	.51
10 percent.	129.0	7.0	.07	.14	.10
15 percent.	130.5	8.5	.025	.038	.031
25 percent.	136.0	8.0	.0048	.0051	.005
Manor loam:					
5 percent..	123.5	11.0	.51	.58	.54
10 percent.	127 5	10.0	.080	.080	.080
15 percent.	128.0	9.0	044	.037	.040
25 percent.	132.0	8.0	.0076	.0030	.0053
Keyport silt loam:					
5 percent.	125.5	10.5	.13	.12	.12
10 percent.	131.5	8.5	.017	.016	.016
15 percent....	133.0	8.0	.024	.029	.026
25 percent.....	132.0	9 0	.00022	.00024	.00023
Tuxedo clay.					
5 percent.	126.0	10.0	.21	.23	.22
10 percent.	131.5	8.5	.032	.056	.044
15 percent. .	131 8	8 4	.019		
25 percent. .	134.0	8.5	.00015		

By using a sealed-in disk, such as unglazed porcelain with a high saturated capillary height, in place of the sieve shown in figure 10, the amount of water held by a soil at equilibrium at various heights can be determined. To increase the effective height of water column, mercury can replace part of the water or a vacuum below or air pressure above may be applied to the specimen. Typical moisture-height curves are shown in Figure 11. Since the surface tension is a function of temperature, these curves vary somewhat with temperature—the surface tension decreases about 2 percent for an increase in temperature of 18F. (10C.).

At a given height, different materials may be at equilibrium with quite different amounts of water: For instance, at a height of 2 ft., 49 percent of water is held by the clay with the same force that 8 percent of water is held by the sand. Due to hysteresis, depending on whether the material is wetting or drying, a given material may hold different amounts of water at the same height H. Thus, from Figure 11, the sand at a height H = 1 ft. may be at equilibrium with moisture contents from 12 to 20 percent at different times depending upon the previous moisture variations.

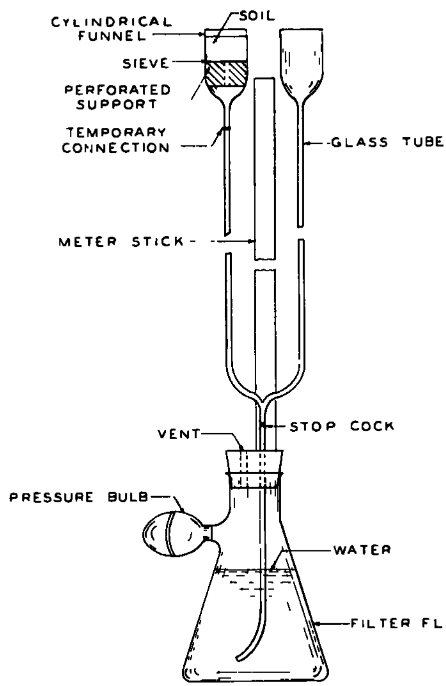


Figure 10. Apparatus for capillarity test.

Specific Yield

The specific yield or volume of water per unit volume of soil removed by drainage is:

$$y = \frac{m_0 - m}{100} \times \frac{w_0}{62.4} \qquad (5)$$

where

m_0 = moisture content before drainage, in percent.

m = average moisture content after drainage, in percent.

w_0 = dry density, in pounds of dry soil per cubic foot of wet soil.

To include water removed from a capillary fringe of height H_1 above the water

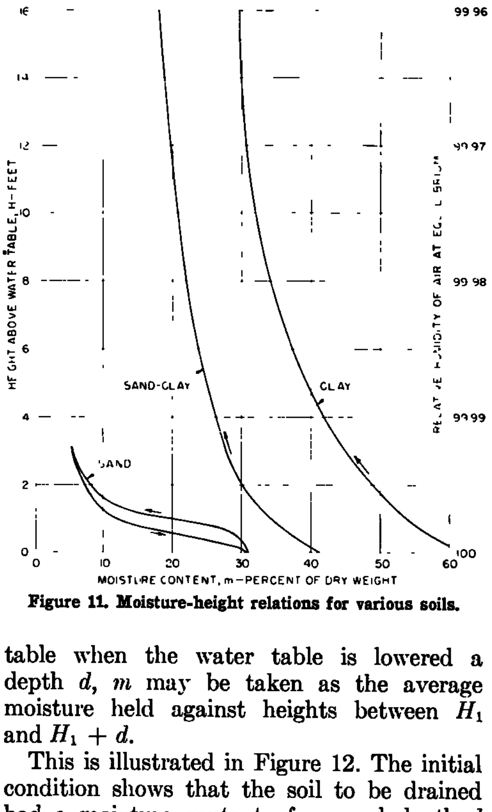


Figure 11. Moisture-height relations for various soils.

table when the water table is lowered a depth d, m may be taken as the average moisture held against heights between H_1 and $H_1 + d$.

This is illustrated in Figure 12. The initial condition shows that the soil to be drained had a moixture content of m_0 and depth d with a capillary fringe above the water table of depth H_1. The final condition shows the water table lowered a depth d with a capillary fringe of depth $H_1 + d$. The capillary fringe of depth H_1 is common to both conditions so that the moisture drained is from m_0 for the initial condition to the average moisture m in the capillary fringe over the distance d from H_1 to $H_1 + d$ above the water table.

For example, consider the sand-clay curve in Figure 11, assuming $H_1 = 1$ and $d = 3$. Here $m_0 = 41$ and the average moisture content from $H_1 = 1$ to $H_1 + d = 4$ is $m = 29$. Assuming $w_0 = 90$, then $y = 0.17$, from Equation 5.

To determine the specific yield of graded sand with various additives passing the No. 200 sieve, the materials represented in Table 7 were tamped in lucite tubes of 1¾-in. inside diameter to a depth of 47 in. at optimum moisture and maximum AASHO density. The samples were supported by a piece of 200-mesh screen wire held by a perforated rubber stopper. In some cases, some water came out of both the top and bottom of the tubes after compaction, apparently due to excess pressure built up in the air trapped in the wet soil. After several days a head of water was applied at the bottom of the tubes, creating an upward gradient until water flowed out the top. The purpose was to saturate the samples but, as shown in the first four lines of Table 10, the air was not readily displaced and saturation could not be accomplished. Thus, for the material with 5-percent limestone added, the compaction moisture was 7.0 percent. This increased to 10.1 percent before drainage, but 13.4 percent was required for saturation.

In one case a vacuum was applied at the top, but this resulted in the entrapment of more air rather than less. While greater saturation could have been obtained by evacuating dry material and then admitting water at one end under a vacuum, this would not simulate field conditions where materials are deposited with water at atmospheric pressure. While marine deposits may be saturated, compacted base courses with appreciable fines do not usually become saturated. For example, a base course (with a plasticity index of 3 and with 6 percent passing the No. 200 sieve) with free water above it and resting on a saturated silty clay subgrade was found to have about 5-percent air voids.

After flow through the samples had been established, they were allowed to drain 10 days. The tubes were then emptied in 2-in. increments, starting from the top, and the moisture content of each increment determined. The results are shown in Table 10. The moisture contents for various heights above the bottom of the tube for the specimens containing 5-percent limestone dust the water table from the surface to various depths, and the values are shown graphically

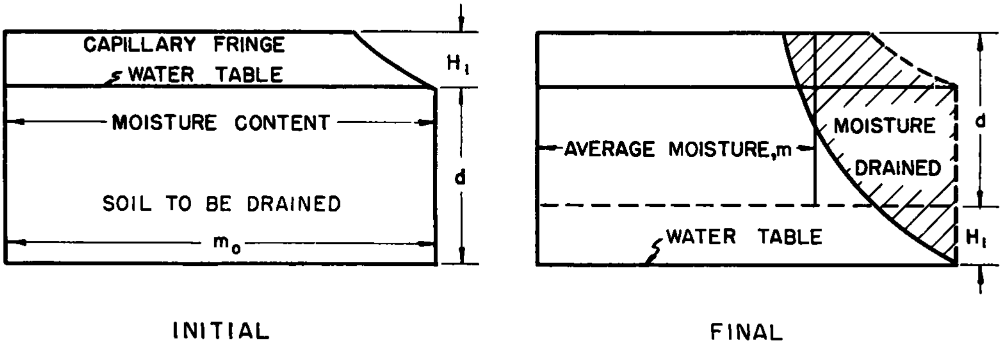


Figure 12. Specific yield including water from capillary fringe.

TABLE 10
MOISTURE RETAINED AFTER DRAINAGE OF SUBMERGED COLUMNS OF SOIL

	Material passing No. 200 sieve added to sand graded from No. 10 to No. 200 sieve[a]									
	Silica		Limestone		Manor loam		Keyport silt loam		Tuxedo clay	
	5 percent	10 percent	5 percent	10 percent	5 percent	10 percent	5 percent	10 percent	5 percent	10 percent
Dry density, lb. per cu. ft.	123	127	123	129	123.5	127.5	125.5	131.5	126.0	131.5
Initial moisture, percent	10 0	9.0	7.0	7.0	11.0	10.0	10.5	8.5	10.0	8.5
Calculated moisture content, saturation, percent	13 4	11.9	13.4	11.1	13.2	11.6	12.5	10.2	12.3	10.2
Moisture before drainage, percent	9 2	9.0	10.1	9.2	10.6	9.9	9.8	8.1	9.6	9.4
Moisture at following heights[b] after drainage, percent:										
46 in.	3.9	6.1	5.1	5.6	5.6	7.7	6.2	6.4	9.5	8.7
44 "	3.9	6.2	5.2	6.0	5.6	7.8	6.2	6.4	9.0	8.0
42 "	4.1	6.0	5.2	5.8	5.7	7.9	6.6	6.4	8.6	9.0
40 "	4.2	6.4	5.4	5.9	6.0	8.2	6.7	6.8	8.7	8.7
38 "	4.4	6.4	5.6	6.7	6.1	8.0	6.9	6.7	8.7	8.6
36 "	4 4	6.6	5.4	6.0	6.3	8.6	7.0	6.9	8.5	8.1
34 "	4.6	6.7	5.4	6.7	6.5	9.2	7.0	7.0	8.8	8.0
32 "	4.8	6.8	5.6	6.0	6.6	9.5	7.0	7.1	9.0	8.3
30 "	5.3	6.7	5.5	6.0	7.0	9.9	7.1	7.7	9.1	8.8
28 "	5.4	6.9	5.7	6.0	7.1	9.8	7.2	8.2	9.3	8.2
26 "	5.9	7.1	5.7	6.0	7.4	9.9	7.0	9.0	8.4	9 4
24 "	6.1	7.3	5.8	6.0	7.6	10.1	7.4	9.0	8.6	9.5
22 "	6.5	7.6	6.0	6.1	7.7	9.7	7.7	9.0	8.2	9.8
20 "	7.1	8.0	6.0	6.6	7.8	9.7	8.1	8.9	8.8	9.3
18 "	7.8	7.9	6.1	6.1	8.2	10.0	8.1	8.8	8.8	10.0
16 "	8.2	8.2	6.2	7 6	8.6	9.7	8.1	8.7	8.6	9.8
14 "	8.3	7.7	6.3	7 1	9.1	9.3	8.5	8.4	8.5	9.9
12 "	8.1	7.9	6.5	7.1	9.0	8.6	9.0	8 4	8.8	10.0
10 "	8.1	7.8	7.2	7.1	8.8	8.6	8.8	8.1	8.9	10.6
8 "	7.9	8.0	7.2	6.7	8.9	8.6	8.9	8.0	9.0	9.8
6 "	8.2	8.2	7.8	7.0	9.2	8.5	9.2	7.4	9.0	10.0
4 "	8.5	8.8	8.5	7.3	9.5	9.1	8.6	7.4	9.2	8.9
2 "	9 1	9.2	9.5	8.2	10.1	9.7	8.8	8.4	9.5	9.1
½ "	11.0	11.2	11.3	11.0	11.6	11.1	9.9	11.1	10.3	11.3
Average moisture after drainage for 10 days, percent	6.4	7.4	6.5	7 1	7.7	9.1	7.7	7.8	8.9	9 4
Specific yield:										
Observed	.055	.033	.071	.043	057	.016	.042	.006	.014	.000
From saturation to drained moisture	.138	.092	136	.083	.109	.051	096	.051	.069	.017

[a] Physical characteristics of admixtures are shown in Table 7.
[b] Center of 2-inch increments except bottom inch.

and 5-percent Keyport silt loam are shown graphically in Figure 13. Using Equation 5, the specific yield was calculated for lowering in Figure 14, assuming initial saturation. For material not initially saturated these values should be reduced by the amount of air voids.

For the full 47-in. depth, Table 10 shows the effect of amount and type of material passing the No. 200 sieve on the specific yield of a well-graded sand.

CAPILLARY FLOW

The curves in Figures 11 and 13 are for a condition of static equilibrium—that is, no flow of water. If water is drawn up to

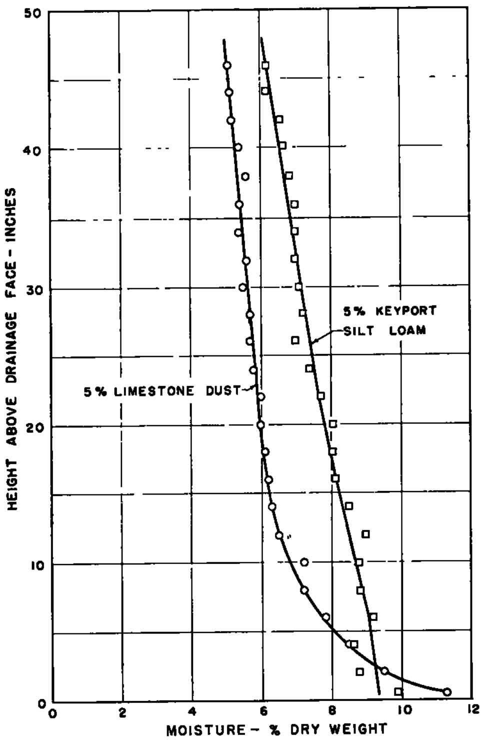


Figure 13. Capillary retention in drained columns of soil.

a given level in the case shown in Figure 13 and removed at a constant rate, as by evaporation or freezing, the moisture content in the soil would decrease until a moisture gradient was established for which this amount of water could be drawn from below the water table. For this condition of dynamic equilibrium or steady state of flow:

$$q = \frac{Q}{A} = k_u\left(\frac{dH}{dz} - 1\right) \qquad (6)$$

where

q = rate of removal of water, in depth (volume per unit area) per unit time.

k_u = coefficient of unsaturated permeability, a function of H.

H = height above water table for static equilibrium corresponding to the moisture content.

z = vertical dimension, positive upward.

The relation between k_u and H has been determined in various ways, such as with permeameters like that shown in Figure 7,

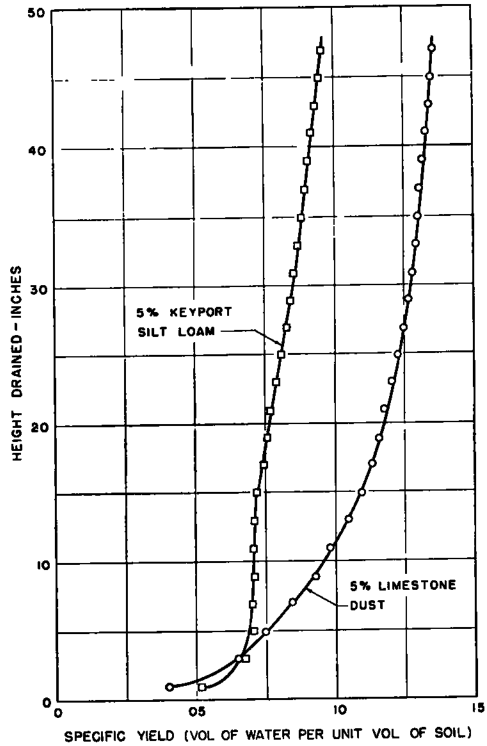


Figure 14. Specific yield for various thicknesses assuming initial saturation.

using negative heads or by measurement of tension gradients in columns of soil subjected to evaporation. Constant temperature is essential because a small change in the moisture content of the soil may account for a large percentage of a small flow One method of determining the tension in the soil moisture is to measure the electrical resistance of a cell buried in the soil. The cell consists of two electrodes separated by an inert porous medium, and is first calibrated under known moisture tensions.

The form of the empirical relation between k_u and H can be approximated over a limited range by the equation: $\log k_u = \log a - H/b$, where a and b are empirical constants.

For the range $H = 1$ to $H = 10$ feet, typical values of a, in feet per day, and b, in feet, as determined by R. E. Moore (*5*), are:

	a	b
Sand	0.28	0.66
Silt	.045	3.9
Light clay	.005	7.5

Substituting the general expression for k_u in equation 6, and integrating, gives:

$$q = a\frac{1 - \text{antilog}\,\dfrac{z - z_0 + H_0 - H}{b}}{\left(\text{antilog}\,\dfrac{z - z_0}{b} - 1\right)\text{antilog}\,\dfrac{H_0}{b}} \qquad (7)$$

For a layer in contact with the water table ($z_0 = H_0 = 0$), the maximum capillary flow upward (infinite H) to a height z is:

$$q = \frac{a}{\text{antilog}\,\dfrac{z}{b} - 1} \qquad (8)$$

or the maximum height for a given rate of flow is:

$$z = b \log\left(\frac{a}{q} + 1\right) \qquad (9)$$

With the values of a and b noted above, this equation gives the values plotted in Figure 15 which shows that, while the clay can lift 1 inch of water per year to the greatest height, the silt gives the maximum height for larger rates comparable to those required for appreciable frost heave. Rates of flow due to capillarity may be much greater than those due to gravity alone. For instance, while the clay (Fig. 15) can lift 100 inches of water per year 0.6 foot, the maximum rate of gravity flow (unit hydraulic gradient) is only 0.005 foot per day (k_u for $H = 0$ which equals a) or 22 in. per year. Similar effects have been noted with portland cement concrete for which the water transmitted by capillarity is much greater than that forced through a sound sample by ordinary pressures.

Effect of Layers

The effect of a layer of sand on the capillary rise of water is shown in Figure 16. (Figs. 16 and 17 show soil cross sections and corresponding moisture content curves). With an impervious surface, so that there is no flow through the surface, as shown on the left side of Figure 16, each material attains the moisture it would hold at a given height if it were continuous to the water table—that is, the sand does not affect the moisture in the silt at static equilibrium. If water evaporates continuously from the surface, as at theright in Figure 16, the soil will dry out enough to

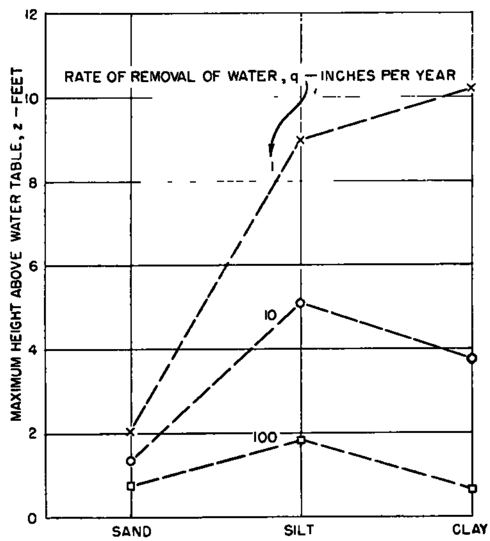


Figure 15. Calculated rates of capillary flow.

establish a tension gradient sufficient to maintain the flow required for continuous operation. The sand causes more drying of the top layer than would occur without it because the unsaturated permeability of the sand at appreciable height above the water table is less than that of silt at this height. If evaporation stops, the moisture increases toward the values for static equilibrium. Upward flow may be satisfactorily limited by a granular layer where there is appreciable evaporation. A buried impervious layer, such as bitumen, above the water table has also been used.

As shown in Figure 17, if water is supplied at the surface, the moisture content is temporarily higher in a silt layer than for conditions of static equilibrium. The presence of a sand layer at appreciable height above the water table retards the downward flow due to its relatively low unsaturated permeability at this height (*6*). For example, a mass of soil in a lysimeter, a box with a perforated bottom separated from the drained undersoil to permit weighing and measurement of percolation, is found to be wetter than the surrounding soil unless a vacuum is applied to its base to replace the moisture tension which the undersoil ordinarily supplies. Thus, for a considerable time after rain ceases the upper layer of silt will be wetter than it would have been without the sand layer. If the water table is kept low in a subgrade, a base on the subgrade may drain better than one placed on a highly porous subbase.

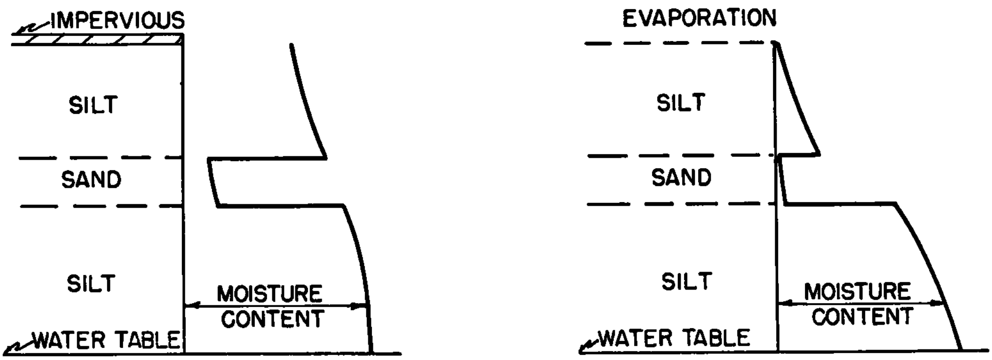


Figure 16. Capillary rise of water.

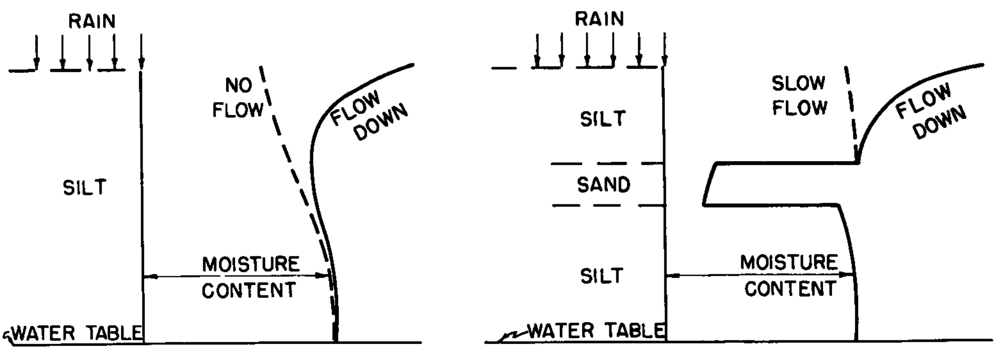


Figure 17. Downward capillary flow.

Other Influences

Temperature difference in the soil causes flow of water since, for equal initial moisture content, water tends to move from a warmer to a cooler area. Another force which causes water to flow is the osmotic pressure such as is caused by differences in concentration of salts at different locations in the soil profile. Flow of soil water may also be caused by an electrical potential. Thus, if two electrodes are placed in a soil and a direct current passed between them, water will flow from the vicinity of the positive electrode to the negative electrode, which could be constructed as a drain. The high cost of the reported field applications which have been made may possibly be overcome by a comprehensive study of this method.

Water may also move through a soil as a vapor. If the air moves as a body, considerable water may be transferred by convection. If the air is still, the vapor may move by diffusion but this is very slow. Since the soil air is generally so nearly saturated, a small decrease in temperature will cause condensation. In some arid regions water has accumulated under pavements, apparently from condensation associated with the rapid cooling of the surface due to radiation under clear skies.

HORIZONTAL PIPE DRAINS

Figure 18 shows the flow of water into buried horizontal drains from a flooded surface, as derived by Kirkham (*7*). The water on the surface could be a film due to rain or water in a permeable base on a much less permeable subgrade. The type of drain is not considered; it is assumed that there is

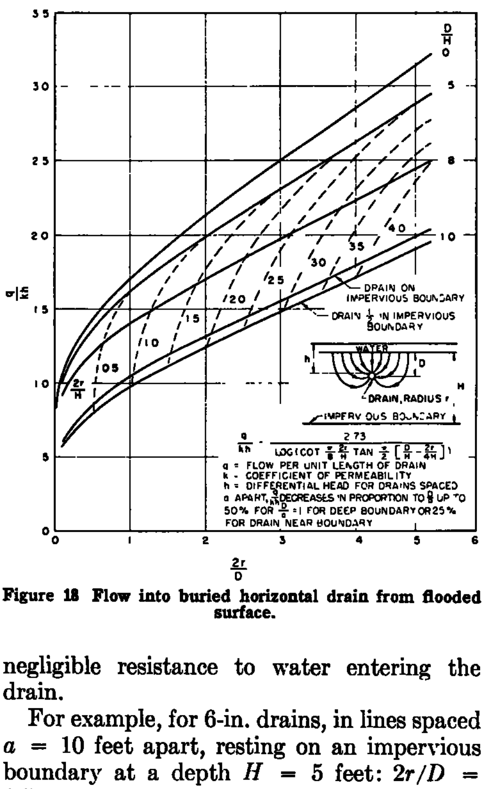


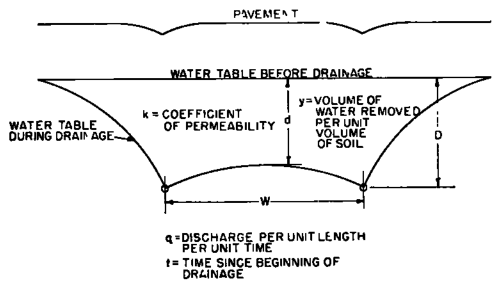
Figure 18. Flow into buried horizontal drain from flooded surface.

negligible resistance to water entering the drain.

For example, for 6-in. drains, in lines spaced $a = 10$ feet apart, resting on an impervious boundary at a depth $H = 5$ feet: $2r/D = 0.5/4.75 = 0.105$, and figure 18 gives $q/kh = 1.05$. For a drain near the impervious boundary, this is reduced by D/a times 25 percent of its value. Thus, the adjusted $q/kh = 1.05 - (4.75/10) \times 0.25 \times 1.05 = 0.93$. For $h = 4.5$, $q = 0.93 \times 4.5k = 4.2k$. For $k = 0.1$ foot per day, $q = 0.42$ cubic foot per day for each foot of length. This is equivalent to $(0.42/10 \times 1) \times 12 = 0.50$ inch per day average infiltration through the surface.

Figure 18 may also be used for computing drainage of a pervious substratum under artesian pressure by inverting the defining sketch shown in the figure. Figure 18 is for a steady state where the flow is continuous with time.

For the unsteady state where the water table is lowering, Figure 19 shows the drainage of a pavement foundation by two small parallel horizontal pipes in the upper part of a deep soil, as determined by McClelland (*8*). Experimentally determined relations between several dimensionless ratios are shown in Figure 19. These ratios may be used to solve various problems, depending upon which values are known or assumed. As an example, take $W = 30$ feet, $D = 3$ feet, $y = 0.1$, and $k = 0.25$ foot per day.



DIMENSIONLESS RATIOS

$\frac{tkD}{yW^2}$	$\frac{d}{D}$	$\frac{q}{kD}$
0 001	0 06	0 80
0 01	0 37	0 47
0 1	0 79	0 25

Figure 19. Drainage by two parallel horizontal pipes.

Suppose the time and rate of flow are desired when d/D reaches 0.79. Then d in Figure 19 is $0.79 \times 3 = 2.4$ feet; $tkD/yW^2 = 0.1$, so that $t = (0.1 \times 0.1 \times 30 \times 30)/(0.25 \times 3) = 12$ days; and $q/kD = 0.25$, giving $q = 0.25 \times 0.25 \times 3 = 0.19$ cubic foot per day per foot. This discharge rate is also the maximum rate of infiltration for which the drains could maintain the drained depth d at 2.4 feet. For $k = 0.0025$ and the same drained depth, q becomes 0.0019 and t becomes 1,200 days, or, for the same time, $t = 12$ days, d/D is 0.06, and d is only 0.18 foot.

BASE COURSE DRAINAGE

If a base course were placed over a relatively impervious subgrade, lateral drainage would be required to drain water entering through the surface. The permeabilities of some bituminous surface mixtures as compacted in the laboratory, previously presented, are higher than those of base course mixtures with an appreciable amount of fines. The effect of traffic and cracks on infiltration needs to be determined. If frost penetrates into the subgrade below a base course, the base is apt to become saturated when thaw occurs from the surface. The frozen subgrade, even though permeable when thawed, may prevent drainage downward so that the water in the base may have to drain laterally to escape.

For a base course on an impervious subgrade which is flooded and then allowed to drain along one edge, the rate of drainage may be approximated (*9*) by the formula:

$$T = US - 0.48S^2 \log (1 + 4.8U/S) \quad \text{for } 0 \leqq U \leqq 0.5, \tag{10a}$$

and

$$T = 0.5S - 0.48S^2 \log (1 + 24./S) + 1.15S \log (S - US + 1.2)/(1 - U)(S + 2.4) \quad \text{for } 0.5 \leqq U \leqq 1 \tag{10b}$$

where

T = kHt/yD^2 = time factor.
k = coefficient of permeability, in feet per day.
H = depth of base, in feet.
t = time, in days.
y = specific yield
D = width of base, in feet.
S = H/Ds = slope factor.
s = cross slope (as 1 percent = 0.01).
U = degree of drainage (as 1 percent = 0.01).

For horizontal base (S = infinity) the equations are:

$$\left.\begin{array}{l} T = 2.4U^2 \text{ for } 0 \leqq U \leqq 0.5 \\ T = 0.6U/(1 - U) \text{ for } 0.5 \leqq U \leqq 1 \end{array}\right\} \tag{11}$$

Curves derived by substituting values in Formulas 10(a, b) are plotted in Figure 20. For U = 50 percent, this relation may be closely approximated by:

$$\left.\begin{array}{l} T = 0.44/(0.74 + 1/S) \text{ or} \\ t = (y/k) \times 0.44D^2/(0.74H + Ds) \end{array}\right\} \tag{12}$$

For example, consider a base course 0.5 foot thick and 20 feet wide with 1-percent cross slope, composed of graded sand with 5-percent limestone dust passing the No. 200 sieve (such as shown in Tables 9 and 10) and placed on an impervious subgrade. For this material k = 0.51 foot per day and, from Figure 14, y = 0.05 between a height of 0 and H + Ds (the lowering of the water table) = 0.5 + 20 × 0.01 = 0.7 foot or 8.4 inches. Assuming, for example, 4-percent air voids before drainage, the time required for 50-percent drainage, from Equation 12, is t = (0.05/0.51) × (0.44 × 20²)/(0.74 × 0.5 + 20 × 0.01) = 30 days.

The results of similar calculations, using Figure 20, for other types of fines and for various slopes, thicknesses, and widths are shown in Table 11, assuming initial saturation and final moisture as shown in Table 10. The relatively short times required for drainage of the mixtures of sand with Tuxedo clay are due to the small yield which means that, even after 50 percent of the drainable water is gone, they are still almost saturated. Comparing these values with the empirical requirement of 50-percent drainage in 10 days suggested by the Corps of Engineers (*10*) indicates that, according to this proposed criterion, graded base courses with as little as 5 percent passing the No. 200 sieve will not drain satisfactorily except for narrow widths. Less densely graded or stratified material may give higher permeabilities, and therefore quicker drainage, even though appreciable material passes the No. 200 sieve. If the materials do not become saturated, it may be that rapid drainage is not necessary. In any event, materials with low plasticity and more than 5 percent passing the No. 200 sieve have often been satisfactory as highway base courses despite slow drainage, even when subjected to frost.

Neglecting entrance losses, the quantity of water transmitted by steady flow through a sloping base course on an impervious subgrade (*9*) is:

$$q = \frac{kH(sD + H/2)}{D} \tag{13}$$

This corresponds to the rate of flow for 42-percent drainage starting from a saturated condition. Thus, roughly, for infiltration through a joint the base would become only 58-percent saturated if drainage were provided at the side. For example, assume a 2-percent slope, H = 0.5 foot, D = 20 feet, and k = 1 foot per day. Then q = [1 ×

0.5(0.4 + 0.25)]/20 = 0.016 cubic foot per day as the maximum quantity that could be transmitted continuously from a joint or crack, any available excess being forced to run off. Over an area of 20 × 1 foot, this quantity of water could be transmitted vertically into a drained subgrade if its permeability were as much as 0.0008 foot per day. If the base had a k value of 100 feet per day, it could transmit 1.6 cubic feet per day which would require a subgrade permeability of 0.08 foot per day for continuous infiltration. While a dense-graded base of low permeability will not transmit water as fast as an open-graded base, it will not have as much total capacity for water, must lose much less water for a given percentage of drainage from a flooded condition, and may make much less water available to the subgrade by its lower infiltration capacity.

TABLE 11

TIME REQUIRED FOR LATERAL DRAINAGE OF 50 PERCENT OF DRAINABLE WATER FOR SATURATED BASE COURSE ON IMPERVIOUS SUBGRADE

Slope, thickness, and width of base course	Time required for drainage with following material added to sand graded from No. 10 to No. 200 sieves									
	Silica dust		Limestone dust		Manor loam		Keyport silt loam		Tuxedo clay	
	5 percent	10 percent	5 percent	10 percent	5 percent	10 percent	5 percent	10 percent	5 percent	10 percent
	days	*days*	*days*	*days*	*days*	*days*	*days*	*days*	*days*	*days*
1-percent slope, 6-inch thickness, and width of										
5 feet	4	16	5	24	3	16	16	69	8	6
10 feet	12	54	16	81	11	54	54	230	25	20
15 feet	25	112	35	130	23	108	108	460	49	41
20 feet	42	185	56	215	38	178	180	760	80	67
25 feet	60	270	82	300	55	255	255	1,090	117	97
30 feet	82	370	114	415	76	345	345	1,475	160	130
1-percent slope, 12-inch thickness, and width of										
5 feet	2	9	3	10	2	8	8	36	4	3
10 feet	8	37	11	40	8	32	32	140	15	12
15 feet	16	78	24	84	16	67	68	290	32	26
20 feet	26	126	39	135	26	108	109	460	53	41
25 feet	39	190	57	200	37	160	160	675	80	60
30 feet	53	260	80	275	52	215	220	920	110	82
2-percent slope, 6-inch thickness, and width of										
5 feet	3	13	4	15	3	14	13	58	6	5
10 feet	10	45	14	52	9	45	44	190	20	17
15 feet	20	94	28	103	20	86	86	370	40	33
20 feet	32	150	45	165	30	135	135	575	63	51
25 feet	45	205	63	225	42	185	185	785	88	70
30 feet	61	270	84	300	55	240	240	1,015	115	90
2-percent slope, 12-inch thickness, and width of										
5 feet	2	9	3	10	2	8	8	36	4	3
10 feet	6	31	10	34	6	27	27	115	14	10
15 feet	13	65	20	68	13	54	56	230	27	20
20 feet	22	107	33	114	22	89	93	380	46	34
25 feet	32	155	49	165	31	130	135	545	66	48
30 feet	44	210	70	230	42	175	185	740	90	65

WATER AND STRENGTH

The presence of water in a base material may decrease its strength in several ways. It reduces the cohesion by lowering the capillary forces; it reduces the friction by reducing the effective weight of the material below the water table; and, for quickly applied loads, it may reduce the strength by the development of pore pressure.

Tests made on sands by other investigators (*11*) showed the bearing capacity to be decreased more than 50 percent due to complete submergence as compared to dry sand. Capillary saturation gave somewhat less reduction. Under dynamic loads even greater loss in strength was obtained. The effects of wetting were especially noticeable for low initial densities of the sand.

Rise of water table in the base also affects the strength of subgrade by reducing the effective pressure on the subgrade. For example, if a 12-inch base and surface course weighing 140 pounds per cubic foot is submerged, the effective weight is reduced to 140 − 62.4 = 77.6 pounds per cubic foot and the effective pressure on the subgrade is reduced from 140 to 77.6 pounds per square foot. According to Figure 21, which shows the

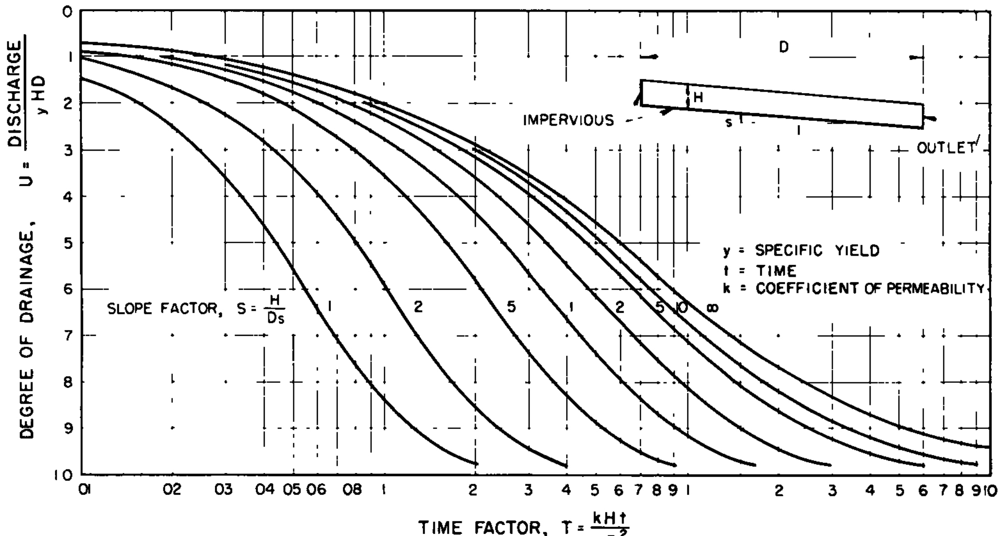


Figure 20. Rate of drainage of flooded base course.

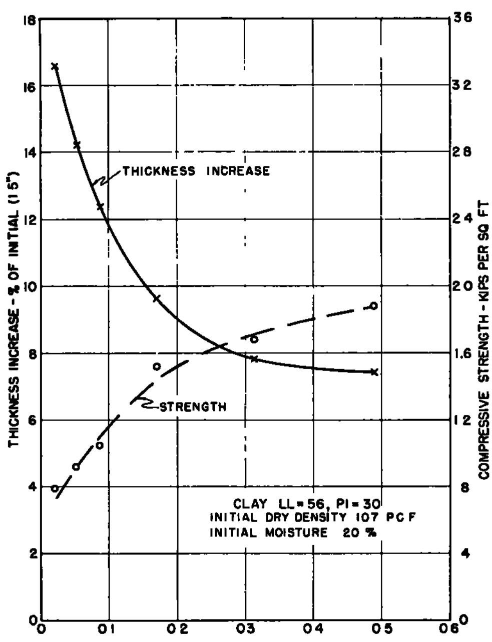


Figure 21. Strength of clay immersed under various surcharges.

strength of a clay at equilibrium under various surcharges, the compressive strength of the subgrade may be reduced from 1.4 to 1.0 kips per square foot by the submergence of the base.

The strength of granular materials under quickly applied loads is greatly affected by density, especially if saturated. This is because the strength depends on the effective stress, which is the total stress minus stress on water or pore pressure, and the pore pressure depends on the density. Thus, if a loose saturated granular material is distorted by shear stresses, the tendency to become denser causes pore pressure (as shown in the upper part of Figure 22), which reduces the effective normal stresses and thereby reduces the strength. On the other hand, a dense material tends to expand when sheared (as shown in the lower part of Figure 22) so that no pore pressure is developed and the total pressure is effective in developing shearing resistance through internal friction.

The fact that some bases appear stable when saturated may be due to the fact that they are dense enough so that they must expand to shear even after a loss of density due to freezing or other causes.

For materials near saturation, freezing may cause a decrease in density due to the expansion of water in freezing, even without ice lenses. Thus, 20-percent water by volume upon freezing increases to 22 percent, causing a 2-percent decrease in density. It has been suggested by C. H. McDonald (*12*) that materials which do not shove during compaction (displace vertically upward at high moisture contents) will be stable when properly compacted regardless of wet and freezing conditions.

While high permeabilities may be obtained by using coarse aggregates, care must be taken to prevent a reduction in their permeability and stability by intrusion of finer soils to which they may be placed adjacent. For instance, if two layers of material are used to make up the test sample in the apparatus shown in Figure 23, and a repetitive load similar to traffic loading is applied by means of the motor and cam, the finer material will be intruded into the pores of the coarser material if the difference in size is too great. Thus, many macadam roads placed on clay have failed when the clay became wet and soft, as indicated in the sketches on the left side of Figure 24. Well-graded aggregate or a fine aggregate subbase, as shown on the right side of Figure 24, will prevent intrusion.

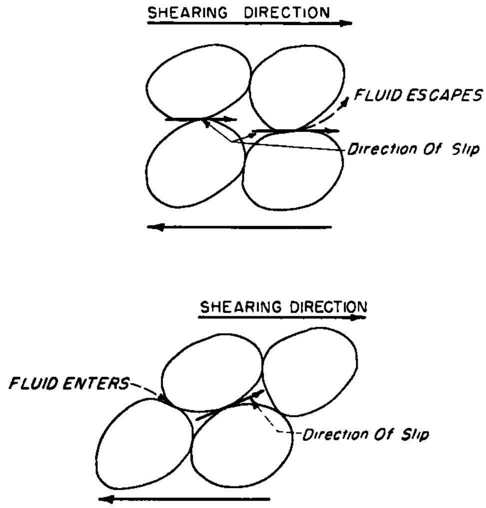


Figure 22. Volume change with shearing.

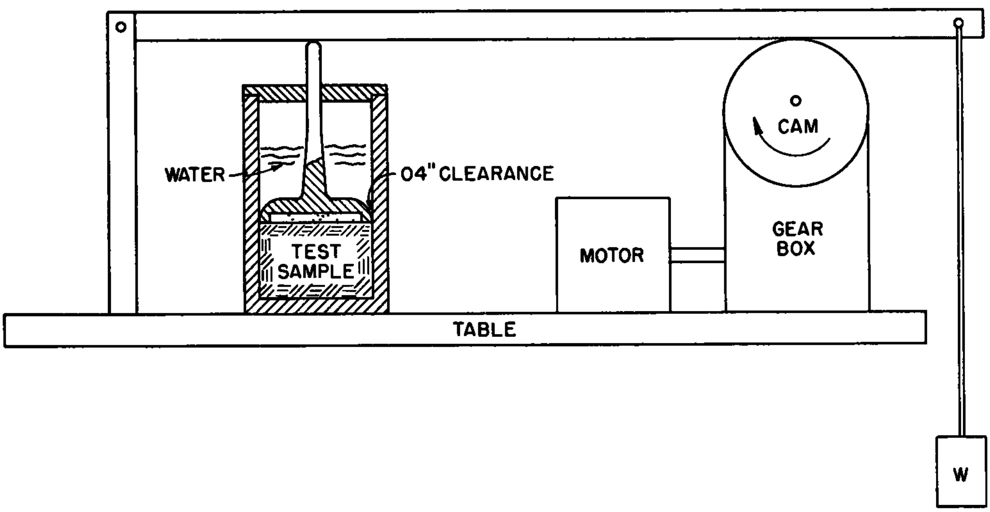


Figure 23. Apparatus for repetitive loading.

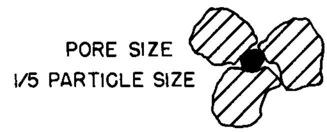
Use of Filters

Subsurface drainage of soil in cut slopes and under pavements is often accomplished by the use of drain tile or perforated pipe placed in a trench and covered with a granular material. The granular material is commonly designated as the filter. If the voids of the filter material are very much larger than the finer grains of the soil to be drained, the fine soil particles are likely to be washed into the pipe or into the interstices of the filter (left and center sketches, Fig. 25), where they accumulate and gradually obstruct the flow.

When the finer particles of the filter material at or near the plane of contact with the soil can hold back the coarser particles of the soil, infiltration or clogging will not be sufficient to materially impede the drainage. Thus, as shown on the right in Figure 25, replacing coarse gravel with sand as backfill in drainage trenches in silt soils prevents the otherwise inevitable intrusion of silt into the gravel. The sand may be kept out of the pipe by putting gravel around open joints or by using perforated pipe with the perforations down.

Granular filters have been used for many years in water filtration and dam drainage. The criterion suggested by Terzaghi and tested and adopted by the Corps of Engineers to prevent intrusion in drain backfills (*10*) is to require that the piping ratio of 15-percent size of filter material to 85-percent size of material to be drained be equal to or less than 5. For plastic clays with sand or silt partings (extremely thin seams) the 15-percent size of the filter—the size than which 15 percent by weight is finer—should be compared to the 85-percent size of the sand or silt. For fractured clays without partings, the 15-percent size of the filter need not be less than 1 millimeter regardless of a smaller value indicated by the piping ratio, because the cohesion of the clay will withstand the seepage forces.

To keep the filter material out of the pipe, the Corps of Engineers requires that the ratio of 85-percent size of filter to pipe opening (perforation or slot) be equal to or greater than 2, which may indicate the desirability of two layers of filter material. To insure adequate permeability in the filter, it is required that the ratio of 15-percent size of filter to



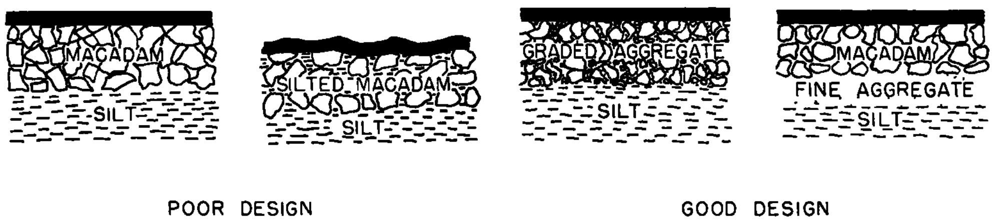


Figure 24. Intrusion of silt into open aggregate.

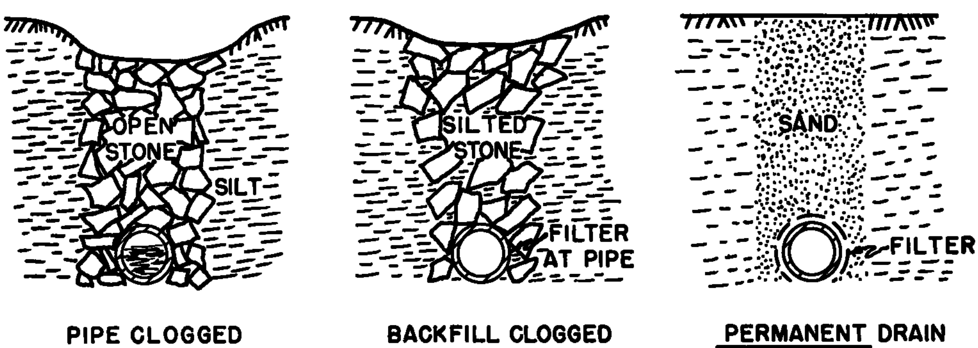


Figure 25. Intrusion of silt into drains.

15-percent size of material to be drained be equal to or greater than 5. The application of this specification is shown in Figure 26. The Connecticut State Highway Department uses

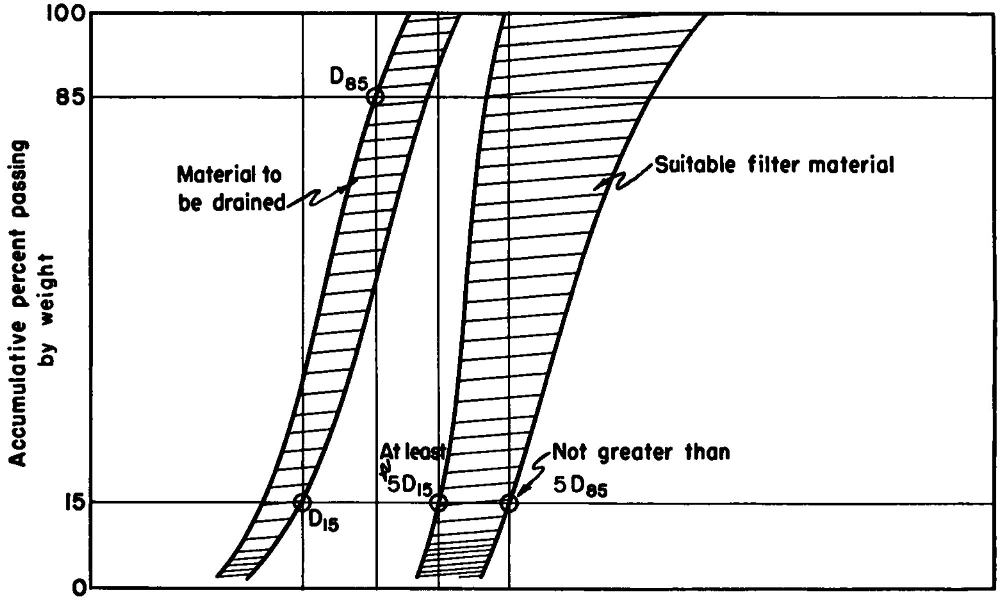


Figure 26. Specification for grain size of material suitable for filter.

TABLE 12
COMPACTION OF GRAVEL MIXTURES UNDER REPETITIVE LOADING

Material, and percentage passing No. 200 sieve	Dry density after— Static load, 16 kips per sq. ft.	Dry density after— 100 repetitions, 4 kips per sq. ft.	Dry density after— 10,000 repetitions, 4 kips per sq. ft.	Thickness reduction—static load to 10,000 repetitions
	Pcf.	*Pcf.*	*Pcf.*	*Percent*
Graded gravel passing ¾-inch sieve: None passing No. 200	112	114	123	9.1
Gravel plus silica dust.				
5 percent passing	117	118	127	7.7
10 percent passing	126	129	136	7.9
15 percent passing	130	134	141	7.9
25 percent passing	130	135	140	5 7
Gravel plus Tuxedo clay·				
5 percent passing	123	125	133	7.3
10 percent passing	130	131	135[a]	5.8
15 percent passing	130	131	132[a]	3 7
25 percent passing	130	130	130[a]	2.1

[a] Density of material below piston; a small amount of pumping occurred.

higher piping ratios which increase with the uniformity coefficient of the soil (*13*).

The exact ratio permissible between subgrade, subbase, and base courses requires further study. The above specification may be satisfactory except that the minimum of 1 millimeter would not apply.

Density of Base Courses

Besides being required for shear strength, densification of base courses is necessary to minimize traffic consolidation which could cause faulting of joints in rigid pavements or uneven surfaces in flexible pavements. It is a problem, when using open-graded materials, to obtain adequate densification with available equipment. By using a homogeneous test sample in the apparatus shown in Figure 23, the densification of materials under repeated loadings has been determined. Table 12 shows the results of such a test on gravel mixtures. The higher initial densities and smaller reductions of thickness under load repetitions is one reason for the use of dense-graded materials for base courses. While the 25-percent admixture gives the least traffic compaction, it has been found necessary to limit the percentage passing the No. 200 sieve to 15 percent and require a plasticity index of not over 6 to prevent softening under wet conditions.

In selecting material for bases under concrete pavements, to prevent pumping, consideration must be given to having enough fines to prevent intrusion of the subgrade soil while not having so much as to cause pumping of the base material itself.

While there is some difference of opinion concerning the need for and the measures of obtaining base drainage, it is generally agreed that thorough compaction is necessary for all types of base material.

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