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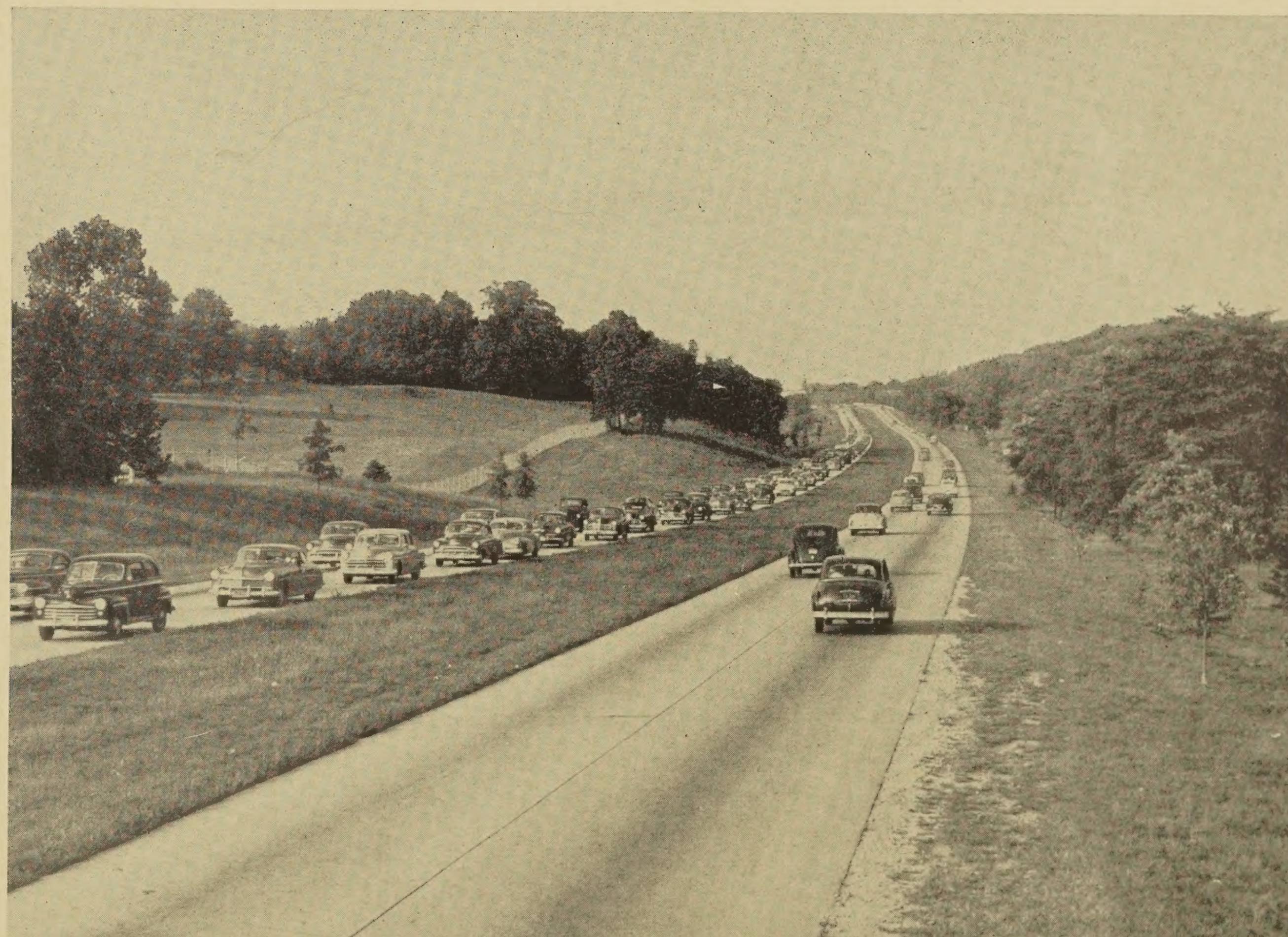

Evening peak traffic on the Shirley Highway

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BUREAU OF PUBLIC ROADS

U. S. DEPARTMENT OF COMMERCE

E. A. STROMBERG, Editor

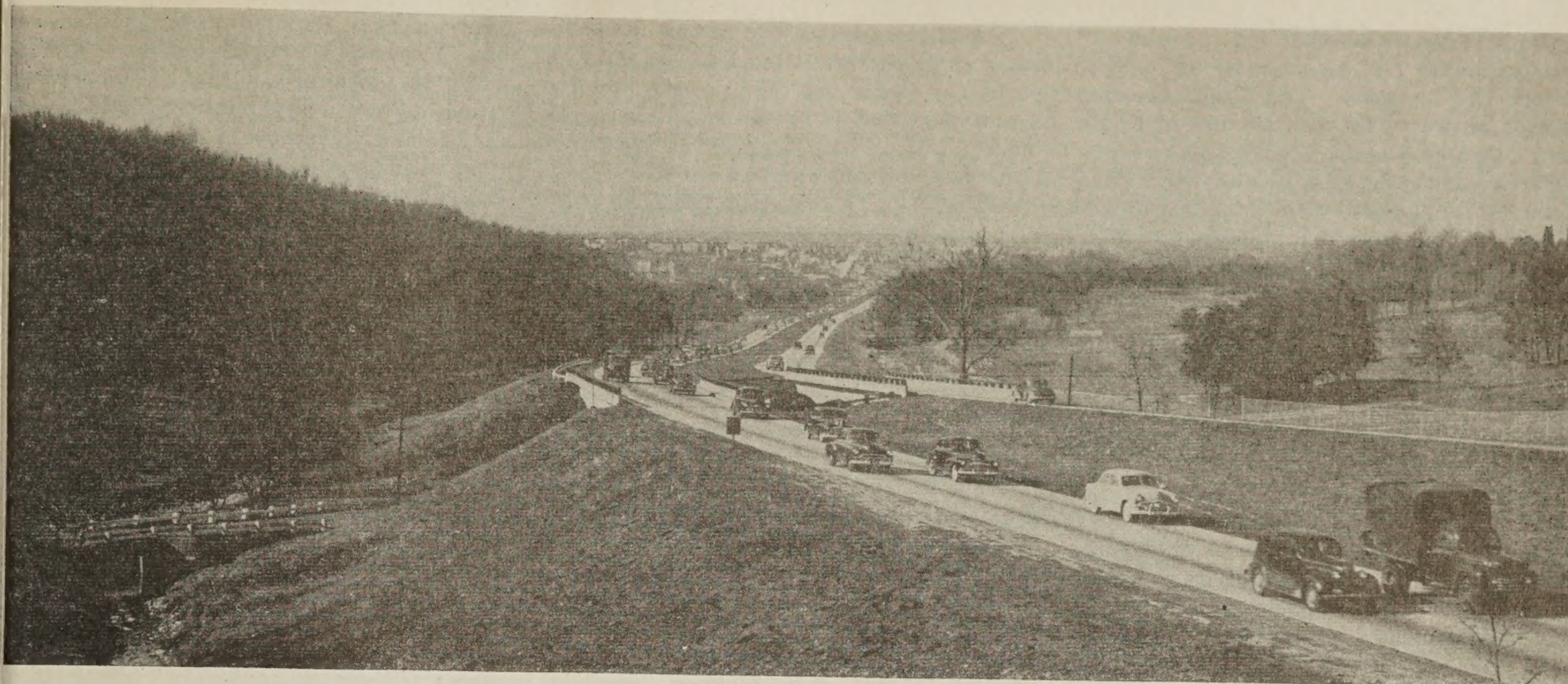


Figure 1.—End of the morning peak traffic on the Shirley Highway.

The Effect of Travel Time and Distance on Freeway Usage

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by DAREL L. TRUEBLOOD
Highway Transport Research Engineer

Until recently, few data have been available concerning the factors that influence motorists in choosing routes of travel in urban areas. Although a number of different factors may be involved, the effect of travel time and travel distance seem especially desirable for initial study because they are items that can be measured with reasonable accuracy on any route and their effect on the action of traffic related to the usage of that route. The relation of these two factors to the usage of the Shirley Highway, a freeway in Arlington and Fairfax Counties, Virginia, has been made a subject of study and the general findings are reported in this paper.

Before definite conclusions, acceptable for wide application, can be reached, the results from this study must be integrated with those from similar studies now underway in other urban areas. In general, though, it appears that motorists regard travel time as more important than distance in choosing a route of travel. Of all the trips on the Shirley Highway that were examined, only 38 percent saved distance while 81 percent saved time.

That motorists are also influenced to some extent by factors other than travel time and distance is evidenced by the fact that 19 percent of the Shirley Highway trips that were examined lost both time and distance by using that highway. Furthermore, of all the trips studied that could have saved both time and distance on the Shirley Highway, 10 percent used an alternate route instead. The reasons why these two groups of motorists chose the routes of travel they did is a question that can only be resolved by additional research.

THE NEED for increased capacity of our urban highway systems is recognized equally by the average citizen and the highway engineer, since both are familiar with the continued increases in vehicles and travel, the growing number of accidents, and the economic loss due to traffic congestion. To be really effective, modernization must be on a scale sufficiently generous to permit the safe, rapid flow of the large volumes of traffic that stream daily into and out of our metropolitan areas and move from point to point within these areas. This requires more than minor improvement of existing inadequate streets. In many instances, new controlled-access expressways to provide increased capacity will be needed.

Accepting this as a premise, the highway engineer charged with the responsibility of planning these new systems is immediately confronted with three questions: (1) What is the capacity of the existing street system? (2) How much additional capacity is needed to serve adequately the present and future over-all traffic demand? (3) What new facilities will be required and what volume of traffic may be expected on them?

Data in the *Highway Capacity Manual* [1] are available for determining an answer to the first question. The second question can be answered through the use of origin and destination traffic study techniques developed during the past 5 or 6 years, when used in conjunction with estimates of future urban growth. The highway engineer is not so fortunate when it comes to answering the third question, however, for he has not been able to estimate with confidence the amount of traffic a new facility will attract from existing streets. Data upon which to base an answer to this question have been lacking. The delay in undertaking research on this subject may be attributed, not to a failure to recognize the need of such information, but rather to a lack of urban expressways upon which data of an empirical nature could be collected.

With attention focused more directly on the improvement and construction of highway transportation facilities in urban areas during the past few years, more projects suitable for this type of research have become available for study. Interest has recently been stimulated through the efforts of the Subcommittee on Factual Surveys of the American Association of State Highway Officials and studies have now been undertaken in several different cities. Such a study was conducted during the summer of 1950 on the urban portion of the Shirley Highway, a freeway in Arlington and Fairfax Counties, Va. The Traffic and Planning Section of the Virginia State Department of Highways assisted in this study by making the field interviews.

Conclusions

Certain general conclusions are revealed from the data collected and analyzed in this study, but these findings must be integrated with those from similar studies now underway in other urban areas before definite conclusions acceptable for wide application can be reached. Considering all of the passenger-car trips between the origins and destinations which might result in freeway usage:

1. A general relation is found between the proportion of trips via the freeway and travel-distance ratios, but the variation in usage of the freeway is quite large when the distance by way of the freeway is approximately equal to or slightly greater than that by an alternate route.

2. Although there is some difference in the proportional use of the freeway for trips of different lengths, the difference does not appear to be greatly significant insofar as traffic assignment is concerned.

3. Good correlation is found between the proportion of trips via the freeway and the ratio of travel time via that route to the time via the most favorable alternate route.

4. A slightly better correlation than any other explored was found between the proportion of trips via the freeway and the actual time that is saved or lost in traveling by way of the freeway as compared with that by an alternate route.

5. Motorists, in traveling from one point to another in the study area, apparently regard travel time as more important than distance in selecting a route of travel. Of all the trips on the freeway that were examined, only 38 percent saved distance by using that route, while 81 percent saved time.

The Problem

The complexity of travel in urban areas is known to all who study city traffic and city planning. Parallel streets offer many alternate routes of travel and motorists in their daily travel do not hesitate to change routes in order to avoid one which has become congested or otherwise unattractive to use. It is common knowledge that they will go considerable distances out of their way in order to reach attractive, free-flowing arterials of modern design.

Origin and destination traffic studies provide information concerning the total number of vehicles passing from one zone to another in urban areas but this knowledge, within itself, is not sufficient. It is essential, for purposes of design and for other reasons, to estimate the number that will be attracted to a new arterial route when it is constructed. The making of such traffic-volume estimates is commonly referred to as traffic assignment. Since the major proportion of the traffic that will use a new route will usually consist of vehicles diverted from the existing street system, the extent to which they can be diverted to the new route and the factors which influence that diversion are of vital importance to those who have the responsibility for planning adequate highway facilities.

In the absence of factual data there is at present some disagreement among highway engineers regarding the reasons a motorist chooses one route instead of another. Consequently there is lack of agreement regarding the proper basis upon which to make traffic assignments. Travel time, travel distance, length of trip, ability to keep moving, safety, convenience, economy, habit, and other factors may enter into the choice. Very little is known, as yet, about the individual effect of any one of these factors. Some engineers consider travel time alone to be the most significant; others believe travel time and travel distance to be equally important; opinions concerning the significance of the other factors are usually indefinite and varied.

Although it is possible that a number of different factors may be involved, travel time and travel distance appear the most promising for initial study because they are measurable items. Both travel time and distance can be determined with reasonable accuracy on any route, even a new one proposed for construction. Furthermore, if a definite relation exists between either one or a combination of these two factors and the choice of routes, that relation, when established, will provide a practicable basis upon which traffic assignments can be made with confidence. It is, therefore, the effect of these two factors on the usage of the Shirley Highway that is explored in this study. The findings reported herein pertain strictly to diverted traffic and are limited to passenger-car travel.

The Henry G. Shirley Memorial Highway extends southwesterly through Arlington and Fairfax Counties in Virginia from a point near the Pentagon. At the north end it connects with a network of expressways serving that building, and, via this network, with three bridges crossing the Potomac River to Washington, D. C. Access to either the Shirley Highway or several alternate routes of travel from any one of the three bridges is readily available by way of this network.

Shirley Highway Selected for Study

The Shirley Highway is a four-lane divided freeway with full control of access throughout its entire length. Each lane is 12 feet wide, and a 30-foot grass median separates the opposing directions of travel. The posted speed limit for passenger cars in Arlington County is 50 miles per hour while in Fairfax County it is 55 miles per hour. Through trucks were prohibited from using the route at the time of this study.

The length of the freeway is approximately 18 miles from its beginning near the Pentagon to the point where it joins U S 1 south of Alexandria. Slightly more than 5 miles at the north end pass through a residential area suitable for a study of the type herein reported. Within the 5-mile section are five traffic interchanges where vehicles may enter or leave the freeway. At the time of this study, the average weekday traffic volume near the middle of the study section was about 30,000 vehicles per day, including both directions of travel.

The cover illustration shows the Shirley Highway, looking north from a point just north of the Glebe Road interchange. This picture, taken in September 1950 at 5:30 p.m., shows the heavy outbound movement of traffic during the evening peak period of travel. Figure 1 is a view in the opposite direction, looking south from the Arlington Ridge Road interchange. This picture was taken in April 1950 about 9 a.m., just after the inbound morning peak had passed. Some of the populous residential area served by the freeway is shown in the background.

There are three principal alternate routes of travel, in addition to the Shirley Highway, which serve the area selected for study. These are the Mount Vernon Me-

[1] *Highway Capacity Manual*, by the Committee on Highway Capacity, Department of Traffic and Operations, Highway Research Board. Published by the Bureau of Public Roads.

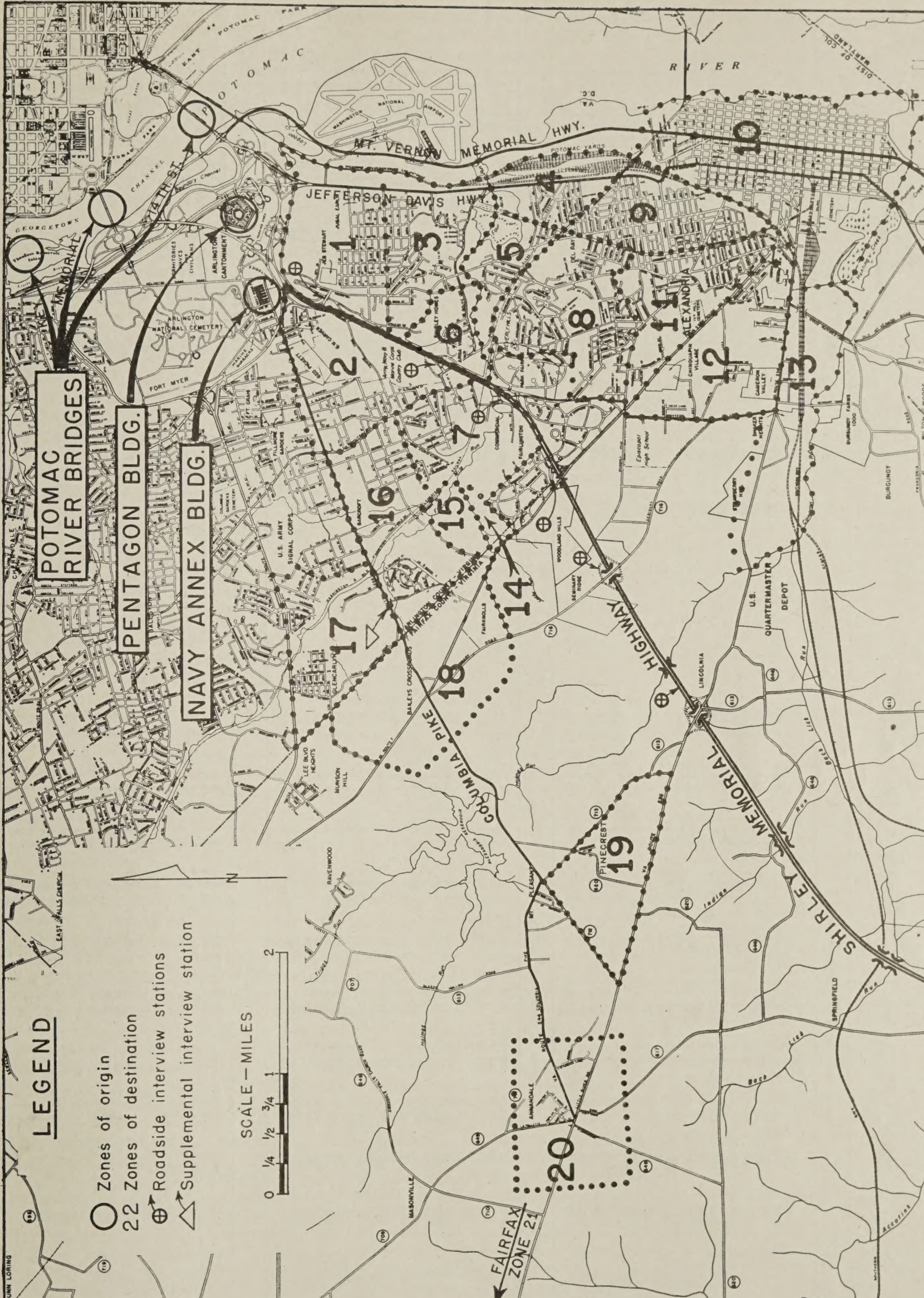


Figure 2.—Map of the study area.

Table 1.—*Origin, destination, travel time, and travel distance of trips studied*

Zone of destination	Number of trips: Total	Via freeway: Number	Via freeway: Percentage of total	Travel time: Via freeway	Travel time: Via alternate	Travel time: Ratio, freeway to alternate	Travel time: Time differential	Travel distance: Via freeway	Travel distance: Via alternate	Travel distance: Ratio, freeway to alternate
Origin at Fourteenth Street Bridge										
			Percent	*Min.*	*Min.*		*Min.*	*Miles*	*Miles*	
1	785	170	21.7	6.3	4.6	1.37	−1.7	2.8	1.9	1.47
2	890	52	5.8	10.3	7.0	1.47	−3.3	4.4	3.3	1.33
3	424	131	30.9	7.0	6.0	1.17	−1.0	3.4	2.4	1.42
4	332	19	5.7	11.0	7.0	1.57	−4.0	5.1	3.1	1.65
5	576	496	86.1	8.0	8.8	.91	.8	3.7	3.2	1.16
6	634	370	58.4	7.3	8.3	.88	1.0	4.0	3.7	1.08
7	1,192	1,172	98.3	6.5	13.5	.48	7.0	3.9	4.9	.80
8	860	478	55.6	9.5	10.0	.95	.5	4.9	4.2	1.17
9	675	148	21.9	12.2	9.9	1.23	−2.3	5.2	4.3	1.21
10	2,308	188	8.1	18.0	13.0	1.38	−5.0	7.7	5.2	1.48
11	467	193	41.3	12.0	13.0	.92	1.0	6.2	4.9	1.27
12	108	100	92.6	12.2	16.4	.74	4.2	6.2	5.9	1.05
13	176	43	24.4	14.0	21.0	.67	(1)	(1)	(1)	(1)
14	57	55	96.5	9.8	17.2	.57	7.4	5.4	6.9	.78
15	89	84	94.4	10.0	17.0	.59	7.0	5.5	6.8	.81
16	700	151	21.6	12.7	10.7	1.19	−2.0	4.9	4.3	1.14
17	177	18	10.2	14.7	11.7	1.26	−3.0	6.9	4.7	1.47
18	322	169	52.5	12.6	12.9	.98	.3	6.4	5.4	1.19
19	72	60	83.3	14.6	18.6	.78	4.0	8.4	8.2	1.02
20	291	196	67.4	17.7	20.2	.88	2.5	10.2	9.4	1.09
21	60	37	61.7	27.3	29.8	.92	2.5	16.3	15.5	1.05
Origin at Memorial Bridge										
1	242	82	33.9	6.8	6.9	0.99	0.1	3.2	3.2	1.00
2	382	22	5.8	10.8	7.4	1.46	−3.4	4.8	3.5	1.37
3	184	84	45.7	7.5	8.3	.90	.8	3.7	3.7	1.00
4	200	15	7.5	11.5	9.2	1.25	−2.3	5.4	4.5	1.20
5	192	123	64.1	8.5	11.1	.77	2.6	4.0	4.6	.87
6	198	156	78.8	7.8	11.3	.69	3.5	4.3	4.6	.93
7	322	321	99.7	7.0	13.9	.50	6.9	4.3	5.3	.81
8	346	284	82.1	10.0	12.3	.81	2.3	5.2	5.5	.95
9	188	62	33.0	12.7	12.1	1.05	−.6	5.6	5.5	1.02
10	560	65	11.6	18.5	15.2	1.22	−3.3	8.1	6.9	1.17
11	65	44	67.7	12.5	15.3	.82	2.8	6.6	6.2	1.06
12	153	105	68.6	13.0	16.5	.79	3.5	6.6	7.2	.92
[2] 13	27	36	...	...	...	...	...	...	...	...
14	28	28	100.0	10.3	17.6	.59	7.3	5.7	7.2	.79
15	27	27	100.0	10.5	17.4	.60	6.9	5.8	7.1	.82
16	406	60	14.8	13.2	11.1	1.19	−2.1	5.3	4.5	1.18
17	101	5	5.0	15.2	12.1	1.26	−3.1	7.1	4.9	1.45
18	377	169	44.8	13.1	13.3	.98	.2	7.2	5.9	1.22
19	43	30	69.8	15.0	15.9	.94	.9	9.2	7.3	1.26
20	281	199	70.8	18.1	20.6	.88	2.5	11.1	9.9	1.12
21	54	49	90.7	27.8	30.3	.92	2.5	17.1	16.0	1.07
Origin at Key Bridge										
1	184	66	35.9	9.3	8.4	1.11	−0.9	4.2	3.9	1.08
2	198	8	4.0	13.3	9.9	1.34	−3.4	5.7	4.4	1.30
3	105	23	21.9	10.0	9.8	1.02	−.2	4.6	4.5	1.02
4	49	10	20.4	14.0	10.8	1.30	−3.2	6.3	5.2	1.21
[2] 5	60	137	...	...	...	...	...	...	...	...
6	86	73	84.9	10.3	12.8	.80	2.5	5.2	5.7	.91
[2] 7	141	157	...	...	...	...	...	...	...	...
[2] 8	2	32	...	...	...	...	...	...	...	...
9	113	57	50.4	15.2	14.7	1.03	−.5	6.5	6.4	1.02
10	290	46	15.9	21.0	17.8	1.18	−3.2	8.9	7.7	1.16
11	65	30	46.2	15.0	16.8	.89	1.8	7.5	7.0	1.07
12	28	18	64.3	15.5	19.0	.82	3.5	7.5	7.7	.97
[2] 13	0	11	...	...	...	...	...	...	...	...
14	20	20	100.0	12.8	18.7	.68	5.9	6.7	7.3	.92
15	24	22	91.7	13.0	18.5	.70	5.5	6.8	7.2	.94
16	115	17	14.8	15.7	12.2	1.29	−3.5	6.1	4.6	1.33
17	119	0	0	17.7	11.0	1.61	−6.7	8.0	5.8	1.38
18	23	11	47.8	15.6	15.8	.99	.2	8.0	6.7	1.19
19	7	4	57.1	17.6	19.2	.92	1.6	10.0	7.3	1.37
[3] 20	...	...	...	...	...	...	...	...	...	...
[3] 21	...	...	...	...	...	...	...	...	...	...
Origin at Pentagon										
			Percent	*Min.*	*Min.*		*Min.*	*Miles*	*Miles*	
1	140	55	39.3	4.0	3.8	1.05	−0.2	1.8	1.5	1.20
2	141	19	13.5	7.8	6.2	1.26	−1.6	3.4	2.4	1.42
3	64	56	87.5	4.8	5.8	.83	1.0	2.3	2.1	1.10
4	29	6	20.7	8.5	7.4	1.15	−1.1	4.1	2.8	1.46
5	234	57	24.4	6.3	7.3	.86	(1)	(1)	(1)	(1)
[2] 6	58	98	...	...	...	...	...	...	...	...
[2] 7	220	424	...	...	...	...	...	...	...	...
8	398	241	60.6	7.9	9.2	.86	1.3	3.9	3.8	1.03
9	75	40	53.3	10.5	9.8	1.07	−.7	4.3	4.0	1.08
10	232	66	28.4	15.1	13.6	1.11	−1.5	6.8	4.8	1.42
[2] 11	0	59	...	...	...	...	...	...	...	...
12	65	46	70.8	11.1	14.3	.78	3.2	5.3	5.5	.96
13	30	21	70.0	12.3	17.0	.72	4.7	5.4	6.9	.78
14	59	59	100.0	7.8	16.4	.48	8.6	4.5	6.0	.75
15	50	50	100.0	8.0	16.2	.49	8.2	4.6	5.9	.78
16	148	69	46.6	9.3	10.0	.93	.7	4.0	3.2	1.25
17	154	15	9.7	14.0	11.3	1.24	−2.7	5.7	3.7	1.54
18	284	169	59.5	10.9	12.1	.90	1.2	5.7	4.5	1.27
19	23	19	82.6	12.9	17.7	.73	4.8	7.6	7.2	1.06
20	139	113	81.3	16.0	19.3	.83	3.3	9.3	8.5	1.09
21	17	15	88.2	24.9	29.5	.84	4.6	15.4	14.6	1.05
Origin at Navy Annex Building										
1	65	54	83.1	3.7	4.9	0.76	1.2	1.7	2.1	0.81
2	123	8	6.5	6.9	4.5	1.53	−2.4	2.9	1.5	1.93
[2] 3	17	28	...	...	...	...	...	...	...	...
4	24	8	33.3	8.3	8.0	1.04	−.3	3.7	3.6	1.03
5	67	41	61.2	6.1	8.3	.73	2.2	2.5	3.8	.66
6	74	65	87.8	5.3	8.0	.66	2.7	2.6	3.2	.81
7	143	140	97.9	5.0	11.0	.45	6.0	2.6	3.1	.84
8	67	53	79.1	7.6	10.7	.71	3.1	3.5	4.0	.88
9	49	39	79.6	10.2	11.7	.87	1.5	3.8	4.7	.81
[2] 10	0	50	...	...	...	...	...	...	...	...
[2] 11	0	15	...	...	...	...	...	...	...	...
[2] 12	0	5	...	...	...	...	...	...	...	...
[2] 13	0	6	...	...	...	...	...	...	...	...
14	7	6	85.7	8.0	13.0	.62	5.0	4.2	4.9	.86
15	11	11	100.0	8.2	12.8	.64	4.6	4.3	4.8	.90
16	112	13	11.6	8.9	7.5	1.19	−1.4	3.6	2.3	1.57
17	65	0	0	13.5	8.3	1.63	−5.2	5.2	2.7	1.93
18	72	14	19.4	10.5	9.6	1.09	−.9	5.4	3.5	1.54
19	2	2	100.0	12.5	15.2	.82	2.7	7.2	6.4	1.13
20	28	15	53.6	15.6	16.8	.93	1.2	8.9	7.7	1.16
21	5	2	40.0	25.3	26.4	.96	1.1	15.0	13.8	1.09

[1] Not included for analysis because percentage of traffic using freeway, when related to travel time ratio, falls far out of general range of other data.

[2] Not used in analysis because of inadequate samples and uncertainties in adjustment of 1948-50 travel.

[3] Insufficient data available to make an estimate of the total zone-to-zone movement.

morial Highway, Jefferson Davis Highway (U S 1), and Columbia Pike. The latter two are typical city-street arterials with the usual signalized intersections, commercial development, and accompanying traffic congestion. The Mount Vernon Memorial Highway, being in the nature of a parkway, is more attractive to travel than the other two. There are, of course, many city streets of lesser importance than the three arterials named that also serve the area.

The map in figure 2 shows the general area of the study and the location of the Shirley Highway in relation to the alternate routes and the city streets serving the area.

Study Procedure

The procedure adopted utilizes the origin and destination data collected in the Washington metropolitan area transportation survey, combined with those obtained from roadside interviews made at points of exit along the Shirley Highway. With these data at hand, supplemented with travel time and distance measurements, it was possible to relate the percentage of traffic using the freeway between certain origins and destinations with the ratio of travel time or distance by way of the freeway to that by an alternate route.

The Washington transportation survey provided information concerning the total number of passenger cars moving from one zone to another regardless of the route traveled. This survey was conducted during the summer and fall of 1948 by the home-interview method, a 5-percent sample of the dwelling units being interviewed.

In order to adjust for the larger volume of traffic in 1950, the zone-to-zone movements from the 1948 survey were uniformly increased by 20 percent. The amount of this increase was estimated from July and August traffic counts made in 1948 and in 1950 at 10 automatic recorder stations in the metropolitan area and, also, from a comparison of the travel in 1948 with that in 1950 between the city of Washington and the Fairlington apartment development. Fairlington is a large residential development, containing about 3,600 dwelling units and housing approximately 12,000 people, located directly on the Shirley Highway at the Arlington-Fairfax county line. Practically all of the dwelling units were occupied in 1948 and also in 1950, so a direct comparison of the traffic data was possible.

An increase of 15.2 percent was found at the recorder stations and an increase of 23.1 percent in the Washington-to-Fairlington traffic. It was decided to give slightly more weight to the latter, and a 20-percent increase was selected as reasonable for the uniform expansion. In addition to this expansion, certain zone-to-zone movements were increased by appropriate supplemental factors to account for unusual changes in population, employment, and commercial development known to have occurred since 1948.

The number of passenger cars using the Shirley Highway in going from one zone to another was determined from data collected at roadside interview stations. Interview stations were established on all exit ramps along the freeway from its beginning near the Pentagon to the end of the study area near the Lincolnia interchange (Virginia Route No. 236). This required five interview stations. At the end of the study area, just north of the Lincolnia interchange, a station was established directly on the Shirley Highway and a sample of all outbound passenger cars passing this point was interviewed. Also, to assist in determining the total travel to some of the outlying zones, a supplemental interview station was established on Columbia Pike. The location of these stations is indicated by distinctive symbols on the map in figure 2.

Each station was operated for a period of 16 hours on a weekday, 6 a.m. until 10 p.m., by an experienced crew of the Traffic and Planning Section of the Virginia Department of Highways. During the time of this study, July 19 to August 3, 1950, an average of 23,249 passenger cars passed the six interview stations along the Shirley Highway in the 16-hour period. Interviews were obtained from the drivers of 15,667 of these vehicles, or about 67 percent.

The data were coded, punched on tabulating cards, and appropriate factors applied by hourly periods to expand the information to an average 24-hour weekday representative of the period of the study. A tabulation was then prepared showing the zone of origin and the zone of destination of all outbound passenger-car drivers using the freeway.

In order to investigate the effect of travel time on the choice of route, it was necessary to determine the time required to travel between points of origin and destination via the freeway and via the alternate routes. A comprehensive travel-time map prepared for the Washington transportation survey provided much useful information in this connection. Check runs by the "floating-car" method were made on the freeway and on the principal alternate routes to test for differences between 1948 and 1950 travel time. The times recorded represent average peak-hour conditions on a weekday and were measured to the center of population of each zone.

As with the travel-time measurements, the distances were measured to the center of population of each zone via the freeway and via the shortest alternate route. In each case the mileage was scaled from a 1:24,000-scale map of the study area. A number of field checks made with a passenger car showed close agreement between the scaled distances and the odometer readings.

The time and distance measurements as well as the traffic volumes between points of origin and destination used in the study are shown in table 1.

Method of Analysis

Since a part of the basic data for this study was derived from a 5-percent sample of travel, it follows that zone-to-zone movements of very low volume are not suitable for use. For this reason, it was decided to consider the city of Washington and its Maryland suburbs as a single zone for purposes of this study. All trips originating therein and destined to zones in the study area must cross one of the three Potomac River bridges designated in figure 2. Thus, for purposes of this analysis, these bridges have been considered as points of origin for all trips beginning on the Washington side of the Potomac River. While information relative to the actual bridge crossed was not available, groups of trips were assigned to the most logical crossing according to their zone of origin and zone of destination.

The Pentagon and the Navy Annex Building are major traffic generators on the Virginia side of the Potomac River and these, in addition to the three bridges—Fourteenth Street, Memorial, and Key—spanning the Potomac River, comprise the five points of origin used in the study.

By reviewing the tabulation of passenger cars that used the freeway it was possible to determine the zones in Arlington and Fairfax Counties that were destinations of a substantial number of vehicles using that facility. Twenty-one such zones were tentatively selected. The findings reported in this article are based on an analysis of the travel from the 5 points of origin to these 21 zones of destination. In total, 105 different groups of trips were examined, but 15 were found to be unsatisfactory for use because of inadequate samples, uncertainties in adjustment of 1948 travel to 1950, or for some other reason, and these movements were disregarded in the analysis. Also disregarded in the analysis were trips originating outside of the Washington metropolitan area, since it was assumed that a majority of these trips would tend to follow marked routes regardless of the attractiveness of such routes for travel. In table 1 it will be noted that a few zone-to-zone movements of low volume were used, this being made possible through the use of the data collected at the supplemental

Table 2.—*Total number of trips studied*

	Number of trips	Percentage of total
		Percent
On freeway	8,152	39.0
On alternate routes	11,604	55.5
Subtotal	19,756	94.5
Not used	1,158	5.5
Total	20,914	100.0

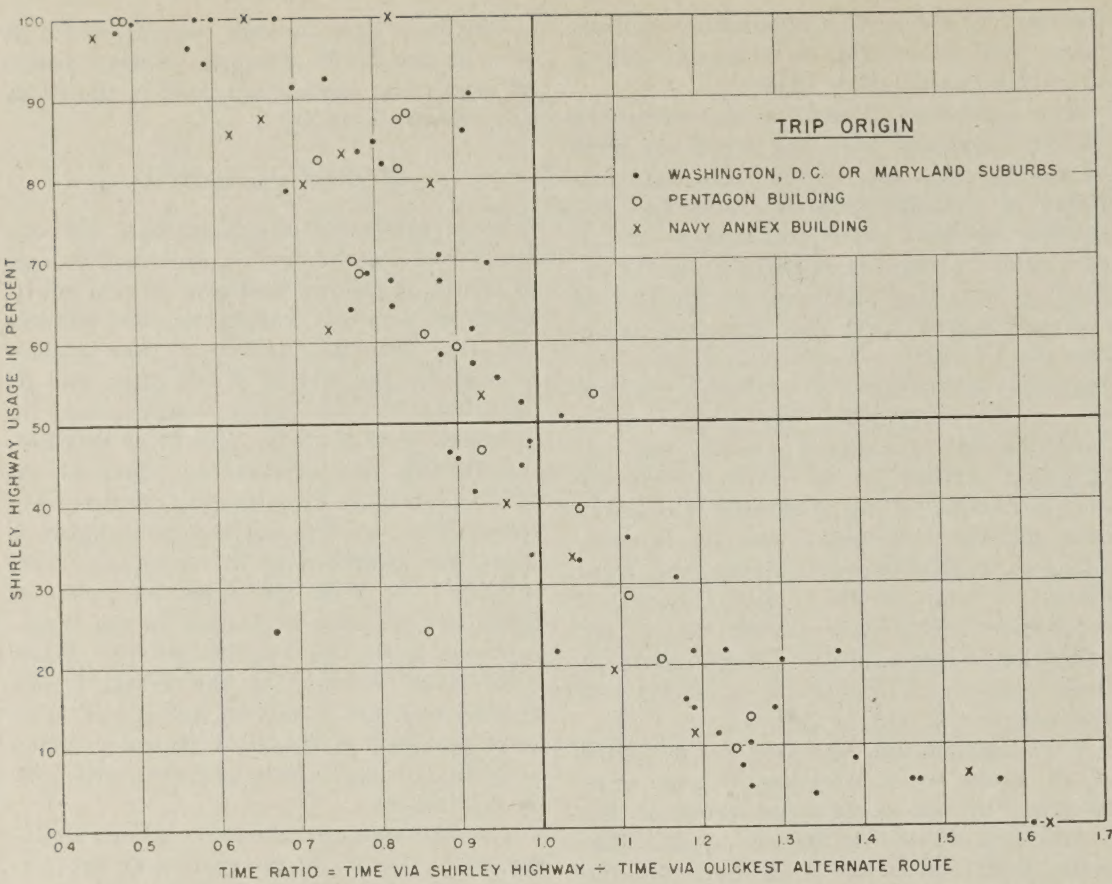


Figure 3.—Freeway usage in relation to time ratio.

roadside interview station on Columbia Pike.

Table 2 summarizes the total number of trips included for study and classifies them according to travel on the freeway, on alternate routes, and those that were not used.

Freeway Use Relation to Travel Time

Figure 3 shows the percentage of passenger-car traffic using the freeway for various travel-time ratios. The travel-time ratio in each case was derived by dividing the amount of time required to make the trip via the freeway by that required via the most favorable alternate route. Each symbol represents the group of trips beginning at one of the 5 points of origin and ending in one of the 21 zones of destination. For example, the small circle near the middle of the chart in the upper right quadrant (1.07 time ratio and 53 percent freeway usage) represents the group of trips beginning at the Pentagon and ending in zone 9. Table 1 shows the total number of trips in this movement to be 75, of which 40 used the Shirley Highway. The dot to the left and slightly below the circle, but also in the upper right quadrant, represents a movement of 113 trips beginning on the Washington side of the Potomac River, crossing Key Bridge and, as it happens, also ending in zone 9. Fifty-seven of these trips used the Shirley Highway.

In total, the 56 dots on the chart represent 16,970 trips originating on the Washington side of the Potomac River, the 18 small circles represent 2,282 trips originating at the Pentagon, and the 16 crosses represent 914 trips originating at the Navy Annex Building. Included are two groups totaling 410 trips that were not used in subsequent analyses because they fall so far out of the general range of the other points. The symbols for these groups are in the 20–30 percent usage of the chart, to the left of 0.9 time ratio.

Although, as expected, there is some scatter in the points, they seem to fall within a reasonably close band all the way across the chart. The general pattern suggests the probability of a relation that may be expressed in terms of an S curve. No attempt was made to fit a curve to the points on this chart, however, because they represent different values insofar as the number of trips is concerned.

To arrive at a weighted mean, and also to reduce the number of points, the data were summarized by combining those movements which have the same travel-time ratio within increments of one-tenth (for example, 0.96 to 1.05) and computing the percentage of the total trips of these combined movements that used the freeway. The results of this summarization are shown by small circles in figure 4. The position of these circles clearly indicates a definite relation between travel-time ratios and freeway usage. While all of the circles do not fall directly on a smooth S curve, especially at each extremity, those near the center fit remarkably well. This may be due, in part, to the greater number of trips represented by those points. The position of the five circles near the center (0.8 to 1.2 time ratios) was determined from a study of 11,205 trips, while the position of the remaining seven circles was determined from 8,551 trips. The curve in figure 4 (and all others shown in this article) was fitted by inspection.

From this curve it is apparent that

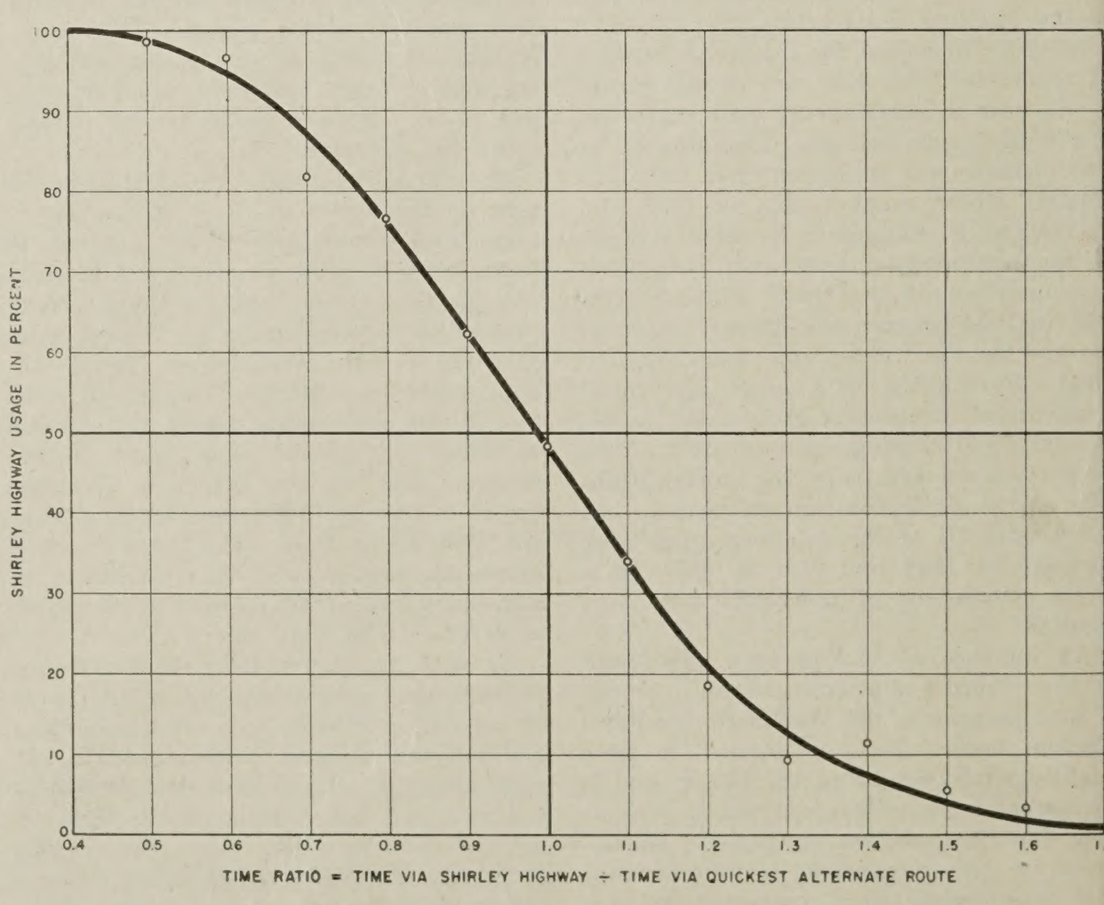


Figure 4.—Curve for freeway usage in relation to time ratio.

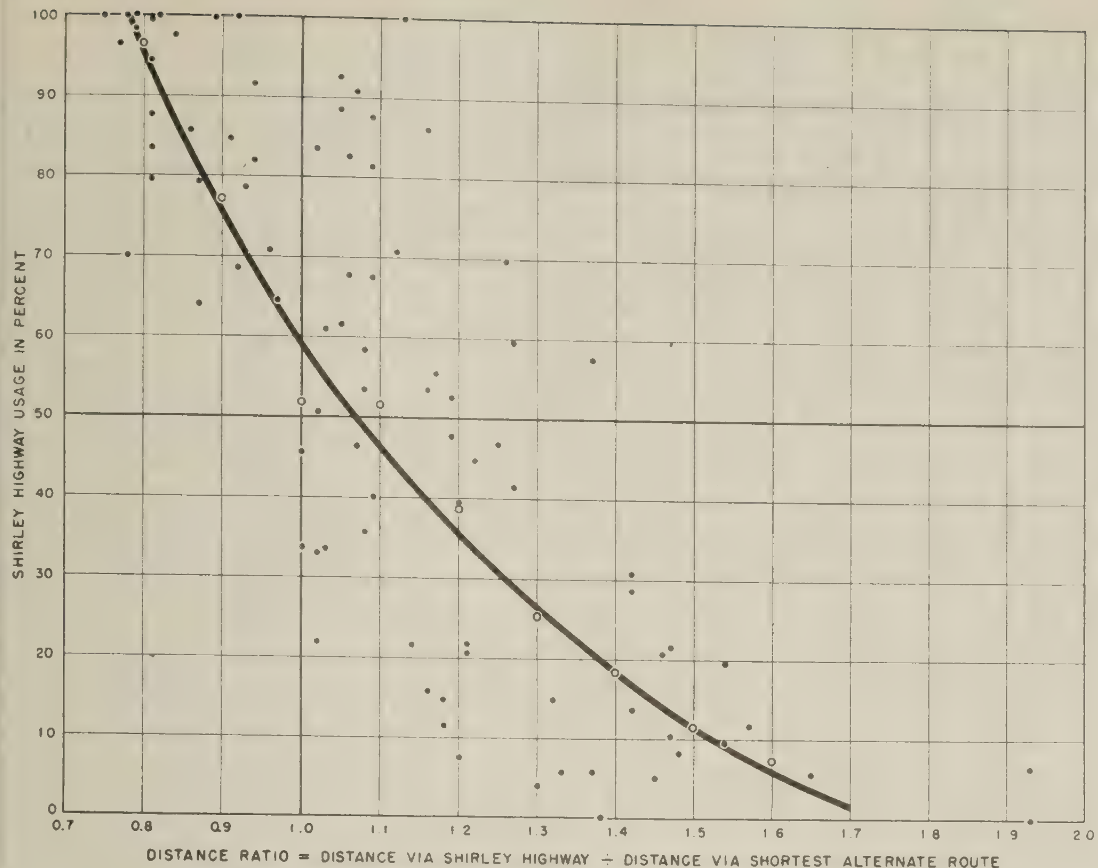


Figure 5.—Freeway usage in relation to distance ratio.

practically all of the motorists use the freeway when the travel time by way of that route is less than 0.4 of that by way of the most favorable alternate route. At the other extreme, when travel time via the freeway is greater than 1.7 times that via an alternate, almost all of the motorists use the alternate route. When the travel time is the same on the freeway as that on an alternate route, approximately 48 percent of the drivers choose the freeway even though it is necessary to travel additional distance in order to do so.

Freeway Use Relation to Distance

Figure 5 shows the percentage of passenger-car traffic using the freeway for various travel-distance ratios. The same general procedure was used in developing this chart as was used for the one shown in figure 4. In this case, however, the scatter of the points is much greater, especially near the middle of the chart between 1.0 and 1.4 distance ratios. Even though weighted means for groups of points with so much variation have little significance, the data were summarized by one-tenth distance ratios (shown by the small circles), and a curve fitted to these circles. Note that the shape of this curve, unlike that of the time-ratio S curve, is concave throughout.

It is evident from the data represented on this chart that practically all of the motorists use the freeway when the distance ratio is less than 0.8 and very few use it when the ratio is greater than 1.7. The usage when the distance ratios are between 1.0 and 1.1 varies from 22 to 92 percent. The exact reason for such a wide variation is unknown, although, from a supplementary analysis, it appears to be directly related to the quality of the traffic service provided by the alternate routes. The 22 movements comprising these trips were separated into two groups—one group having a choice of the freeway or an alternate providing reasonably good traffic service, and another group that must choose between the freeway or a relatively poor alternate. Of the first group, only 37.1 percent chose the freeway, while 66.6 percent of the second group chose that route. Furthermore, all except two of the eight movements included with the first group could travel via alternate routes in the same or less time than via the freeway, while all except one of the fourteen movements included with the second group could save time by using the freeway. Thus it is apparent that motorists making trips that are approximately equal in distance by the freeway and by an alternate route choose the former in greater proportions when travel time can be saved by doing so.

Freeway Use Relation to Time and Distance Combined

Since both the travel-time ratio and the distance ratio appear to bear some relation to the use of the freeway, it was decided to investigate a combination of the two. With this in mind, the distance ratio was divided by the time ratio for each group of trips, in effect giving a speed ratio, and the result plotted according to the percentage of passenger-car traffic using the freeway in each case. No correlation was found with this procedure. A second attempt was made to combine the two ratios, in which the time ratio and the distance ratio for each group of trips were multiplied and the product plotted according to the percentage of passenger-car traffic using the freeway in each case. Figure 6 shows the results of this combination after the detailed data were summarized by increments of one-tenth.

The tendency is more toward a straight line than the S curve found in connection with the time ratio (fig. 4). This is to be expected because, as a matter of mathematics, the product of the time and distance ratios tends to drop the relative position of the product curve below that of the time-ratio curve for each group of trips having a time ratio and a distance ratio both less than 1.0. Conversely, the tendency is to raise the relative position where either or both ratios are greater than 1.0.

While a relation between the freeway usage and the travel time-distance ratio product seems to exist, the correlation is not as good as that found with the time ratio alone. The relation shown in figure 6 is of general interest, but it appears to be less practicable and would provide less accurate results than the time-ratio curve if used as a basis for making traffic assignments.

Freeway Use Relation to Time Differential

Figure 7 shows the percentage of passenger-car traffic using the freeway based on the actual number of minutes motorists saved or lost by using that route as compared with an alternate. Here, as in the case of the travel-time ratio, the points fall within a reasonably close band which unmistakably suggests an S-curve relation.

The curve shown was drawn to fit the weighted means computed for each minute saved and each minute lost. As on previous charts, the weighted means are indicated by small circles. The resulting curve shows that where any group of motorists can save 8 or more minutes by using the freeway, they all choose that route. At the other extreme, a few motorists use the freeway even though they lose 4 or 5 minutes by doing so. When travel time via the freeway is the same as that via an alternate route, the curve shows that approximately 48 percent of the motorists choose the freeway. This agrees properly with the percentage use shown by the time-ratio curve when the travel times are equal.

An interesting feature of this relation is its tendency to group zone-to-zone movements according to length. The longer trips tend to fall near the extremities of the curve while the shorter trips are grouped nearer the middle. This is readily understandable, because it would be impossible to save or lose several minutes by using the freeway instead of an alternate

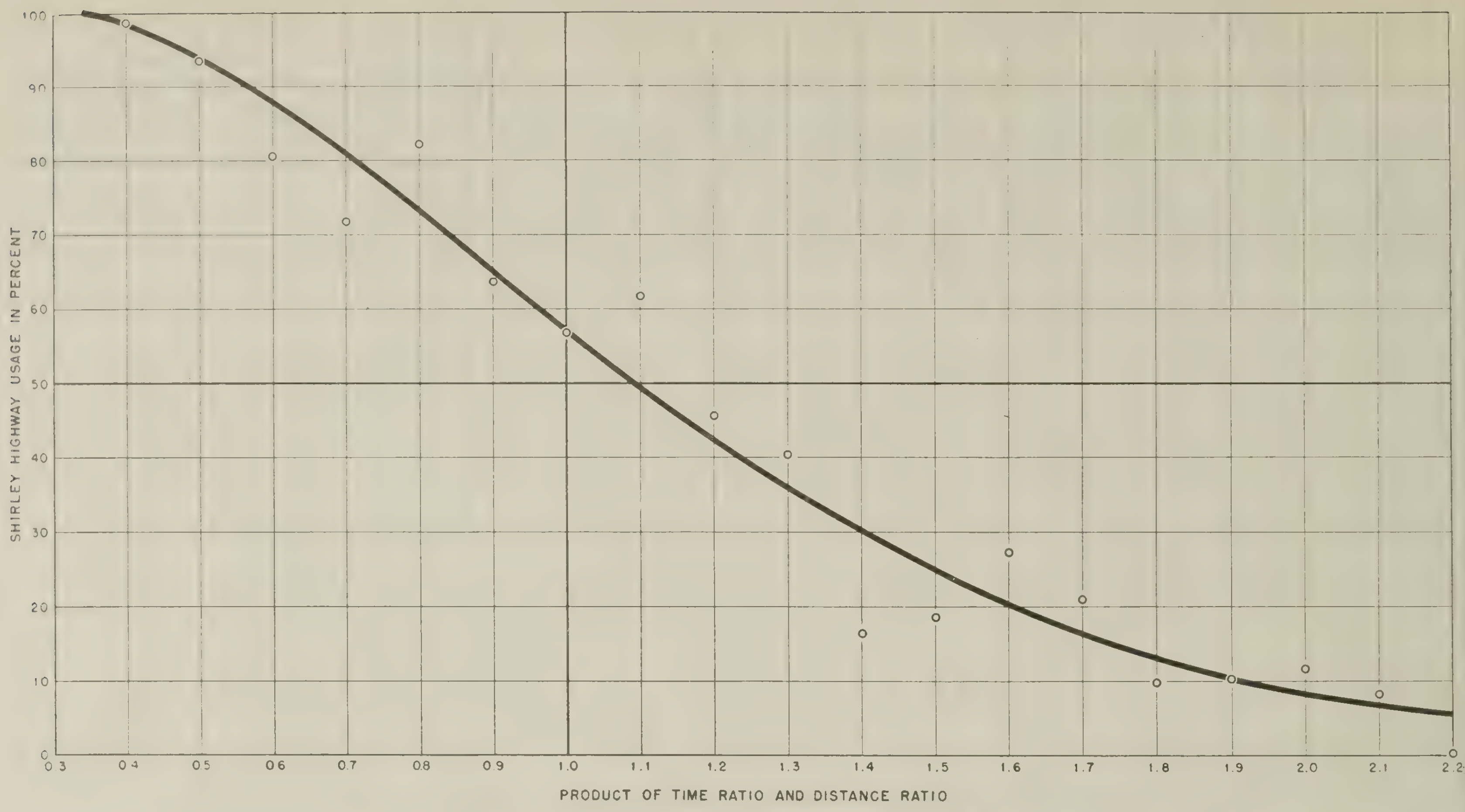


Figure 6.—Freeway usage in relation to product of time and distance ratios.

route in making short trips of only 5 or 10 minutes total duration. On the other hand, in making trips of 20 or 30 minutes duration, a time differential of several minutes would not be at all unlikely.

It is this tendency of trips to fall into groups according to length that results in somewhat better correlation between freeway usage and time differential than between freeway usage and time ratio. The reason for this difference is brought out in figure 8.

Freeway Use in Relation to Trip Length

Figure 8 shows the percentage of passenger-car traffic using the freeway, based on travel-time ratios, for three increments

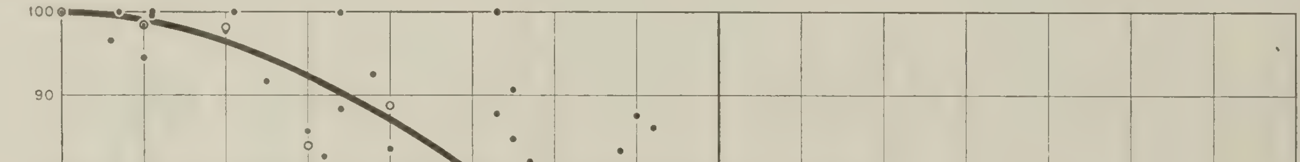


Figure 7.—Freeway usage in relation to time differential.

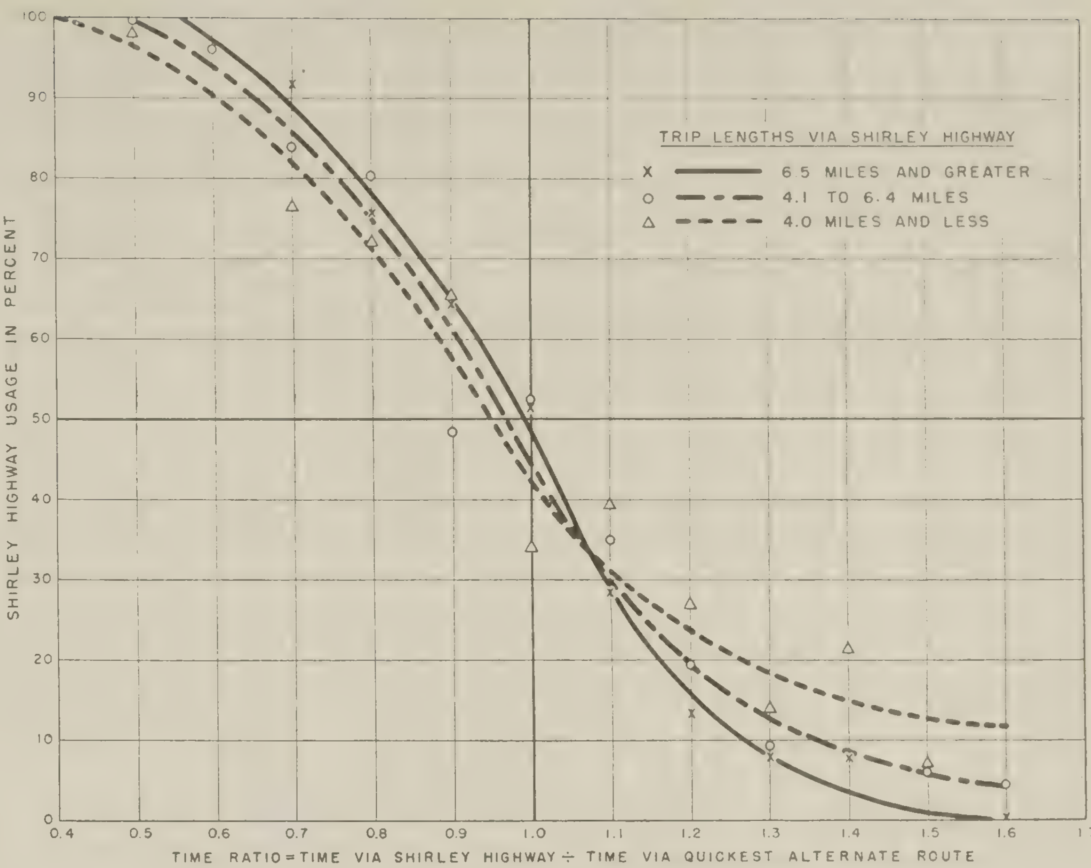


Figure 8.—Effect of trip length on freeway usage.

of travel distance: 4.0 miles and less, 4.1 to 6.4 miles, and 6.5 miles and greater. The distance by way of the freeway was used in grouping the trips into the three increments of length. The length in each case is the over-all distance between one of the five points of origin and one of the zones of destination. On this basis, the shortest trip included is 1.7 miles while the longest is 17.1 miles.

It is evident from the position of the three curves in figure 8 that, when the time ratio is less than 1.07, a greater percentage of the longer trips than of the shorter trips are on the freeway. When the time ratio is greater than 1.07, however, the position of the curves is reversed and a larger percentage of the shorter trips are on the freeway. For example, when the travel-time ratio is 0.7, these curves show that 89 percent of the longer trips are on the freeway and only 82 percent of the shorter ones. When the time ratio is 1.4, only 3 percent of the longer trips are on the freeway but there are 15 percent of the shorter ones.

The explanation for this relation appears to be directly connected with the actual amount of time motorists can save, or will lose, in making trips of various lengths by one route as compared with that of another. This point can best be explained by an example. Assume a long trip to require 20 minutes by way of the freeway and a short one 5 minutes. If the time ratio is 0.7, motorists making the longer trip save 8.6 minutes by using the freeway while those making the shorter trip save only 2.1 minutes. The actual amount of time saved in the case of the longer trip is four times as great as that for the shorter trip. When the time ratio is 1.4, however, motorists lose 5.7 minutes in making the longer trip by way of the freeway, but only 1.4 minutes for the shorter one. In this case the loss in time is about four times as great for the longer trip.

Thus it seems that motorists attach significance to the actual amount of time saved or lost in traveling from one point to another in urban areas (especially when the amount is substantial) as well as to the relative travel time by way of one route compared with that of another. It is quite possible, in the case of the shorter trips, that the increment of time saved or lost is so small that it is not only insignificant but probably unknown to motorists. This might further explain the reason for the relative position of the curves in figure 8.

If the travel-time ratio were the only criterion, the point at which the curves in figure 8 cross each other would occur at a ratio of 1.0 instead of 1.07. The positions of the curves show that, when the travel-time ratio is 1.0, the freeway is slightly more attractive to motorists making long trips than it is to those making short trips. The difference is so small in this case, however, that it could not be considered significant insofar as traffic assignment is concerned.

Freeway Use in Relation to Time and Distance Ratios

The percentage use of the freeway in relation to travel-time ratios and to travel-distance ratios has been shown on charts, separately, in figures 4 and 5. In figure 9 these two ratios and the percentage use of the freeway are shown on the same chart in order that the general relation of the three variables can be visualized and ex-

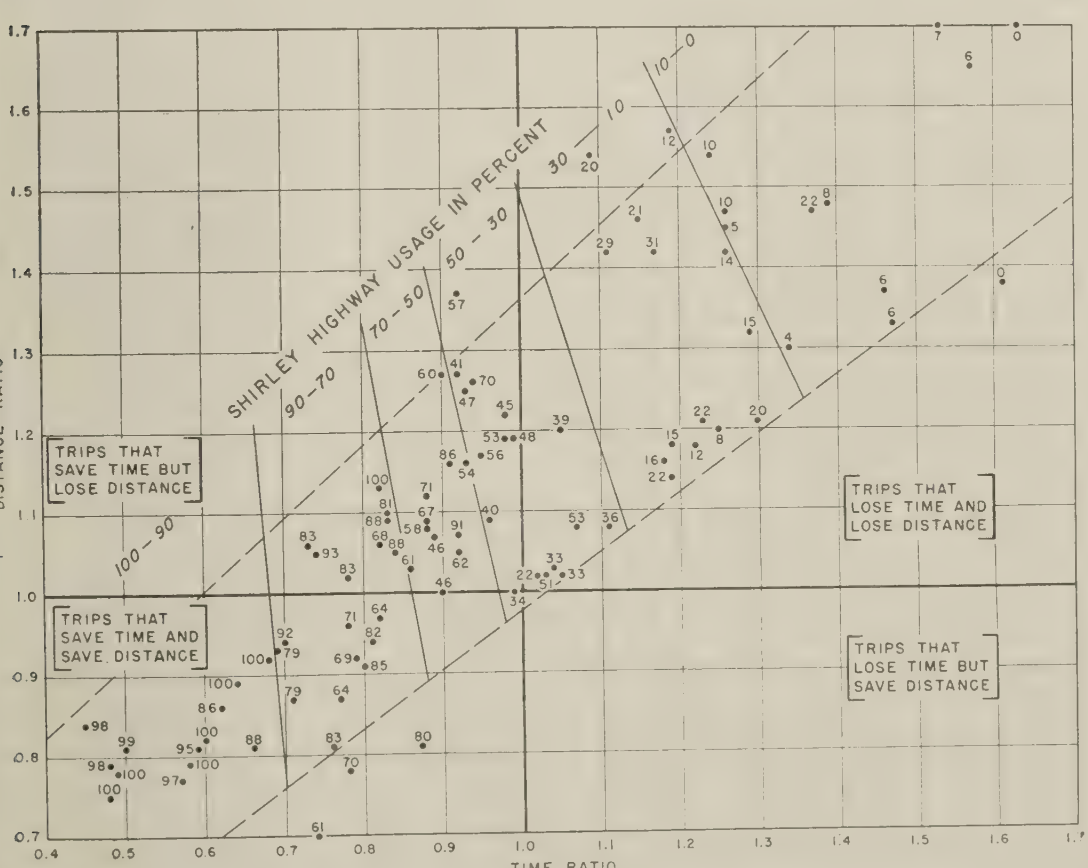


Figure 9.—Freeway usage in relation to time and distance ratios.

plored. Each dot on the chart represents a zone-to-zone movement and the adjacent numeral indicates the percentage of that movement using the freeway. These are plotted according to the time and distance ratios for each such movement.

The four statements shown in brackets on the chart, relative to saving or losing time and distance, apply to the four quadrants formed by the heavy vertical line at time ratio 1.0 and the heavy horizontal line at distance ratio 1.0. These statements refer to trips made by way of the freeway. Note that the lower right quadrant does not contain any dots. This is proper because, in this study, the average speed of travel on the freeway exceeds that on any alternate route; consequently, any zone-to-zone movement that would have lost time on the freeway would also have lost distance.

It is of interest that, in total, the freeway was used by 17 percent of the zone-to-zone movements plotted in the upper right quadrant, by 60 percent of those plotted in the upper left quadrant, and by 90 percent of those plotted in the lower left quadrant. Interpreting these percentages further, of the motorists whose trips were studied that would have lost both time and distance by using the freeway, 17 percent chose to do so, as did 60 percent of those who would have saved time but lost distance. On the other hand, of the motorists that could have saved both time and distance by using the freeway, 10 percent did not do so. This, again, seems to indicate the presence of factors other than time and distance that influence motorists in their choice of route.

The two dashed lines extending from the lower left to the upper right of the chart indicate the general range of time and distance ratios within which usage of the freeway occurs. The five solid lines sloping upward slightly to the left subdivide the area between the dashed lines into six segments. Each segment represents roughly a certain percentage range for use of the freeway as designated by the line of numerals extending diagonally across the chart above the upper dashed line, most of the percentages within a segment falling within the range indicated. It will be noted that the percentage of use gradually decreases from 100 percent at the lower left corner to zero at the upper right corner.

While it would have been desirable to have had more points from which to determine the slope of these five "contour" lines, the general direction of the third and fifth line from the left can be determined with reasonable accuracy from the points shown. To determine the slope of the three remaining lines, the third and fifth were extended to an intersection at a point above the chart and the remaining three lines projected back from that point of intersection as radii of a circle. This method seemed to conform with the data as nearly as any other logical one.

The slope of the resulting lines permits some interesting conjectures to be made. If all had turned out to be vertical this would have indicated that distance ratio has no effect at all on a motorist in his choice of route insofar as the factors of time and distance ratio are concerned. Conversely, had the lines assumed a horizontal position, it would indicate that time ratio has no effect. The lines as drawn suggest that both ratios affect the choice of route to some extent but, since the lines are more nearly vertical than horizontal, it follows that the time ratio is probably more significant than the distance ratio in this respect. Furthermore, since the slope of each line becomes greater as the percentage use of the freeway decreases, it suggests an increasing effect of the distance ratio as the time and distance ratios increase.

Statistical Comparison of Curves

As stated earlier, the principal purpose of this study is to show how travel time and travel distance affect the use of a freeway. The curves developed show the effects of these factors, but the correlation is not perfect in any of these cases. The points in some instances depart widely from the average relation expressed by the trend lines or curves fitted to the data. It is desirable to know the relative significance of the averages expressed by each curve before they can be used intelligently.

The standard error of estimate, or, more briefly, the standard error, offers a mathematical means of making this determination. The standard error serves not only as a general index of the significance of these curves, but also as a measure of the degree of accuracy of estimates based upon them. In other words, it measures the expected variability of estimated values from the actual values.

Therefore, in order to compare the curves developed in connection with time and distance ratios, and appraise their reliability for use in traffic assignment work, the standard error was computed for each curve. The results of these computations, which is the percentage variation that would not be exceeded more often than about one-third of the time, are summarized in table 3.

Of the four curves, the one based on time differential has the least standard error, while the one based on distance ratio has the greatest. It will be noted that the curve based on time ratio has a standard error only slightly greater than that of the time differential curve. This clearly indicates that the curves based on time differential and time ratio are approximately of equal reliability and that time differential and time ratio show the best correlation with the percentage use of the freeway. Either of these curves, if used for purposes of assigning zone-to-zone movements of traffic to the freeway, would provide results within 8 or 9 percent of the true values in at least two-thirds of the cases. This is satisfactorily within the accuracy of the basic data collected in origin-and-destination traffic studies conducted on the usual sampling basis. Moreover, the necessity of projecting traffic estimates into the future, with the attendant uncertainties, can readily introduce differences of greater magnitude than those that would result from the assignment of traffic on the basis of the time differential or travel-time ratio curves.

Table 3.—*Standard error of estimate*

Description of curve	Figure No.	Standard error
		Percent
Time ratio	4	8.66
Distance ratio	5	17.54
Product of time and distance ratios	6	11.14
Time differential	7	8.50

Highway Subdrainage

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by E. S. BARBER and C. L. SAWYER,
Highway Engineers

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.

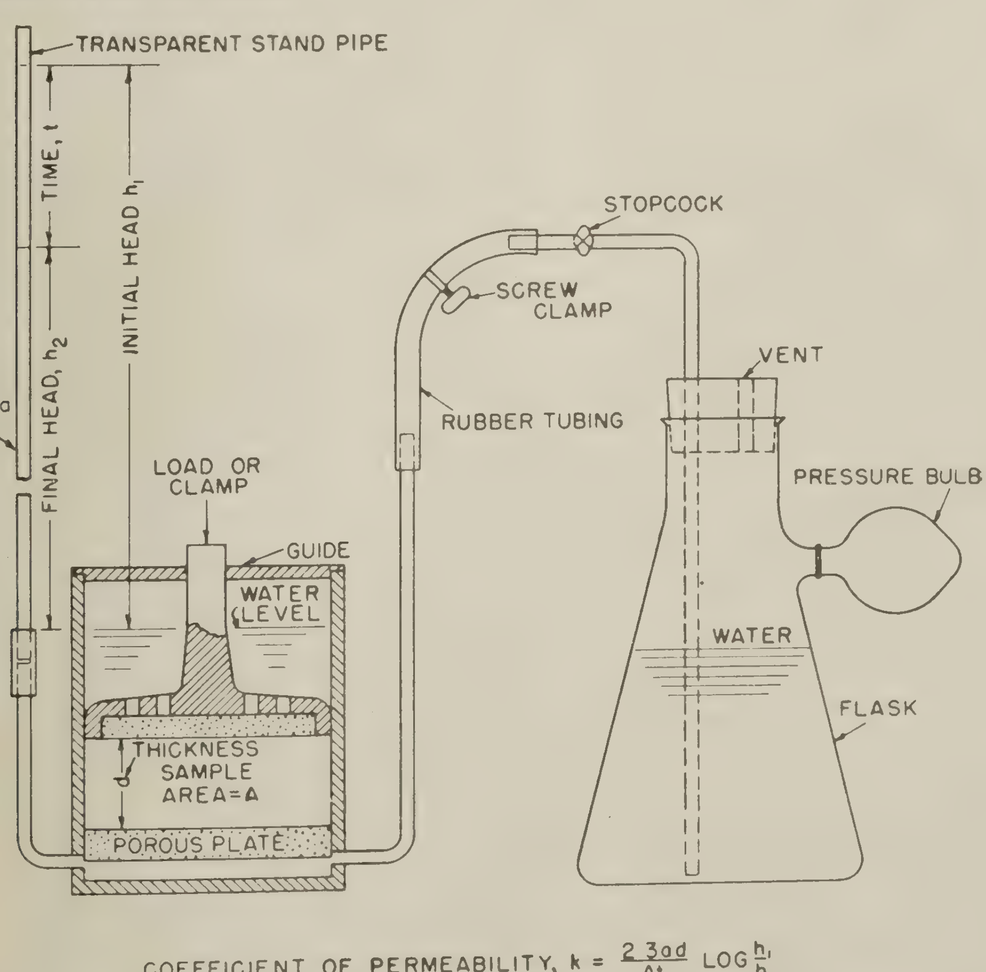


Figure 1.—Apparatus for measuring permeability by means of a falling head.

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:

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

where

Q=volume of flow per unit time.
k=coefficient 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:

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

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,

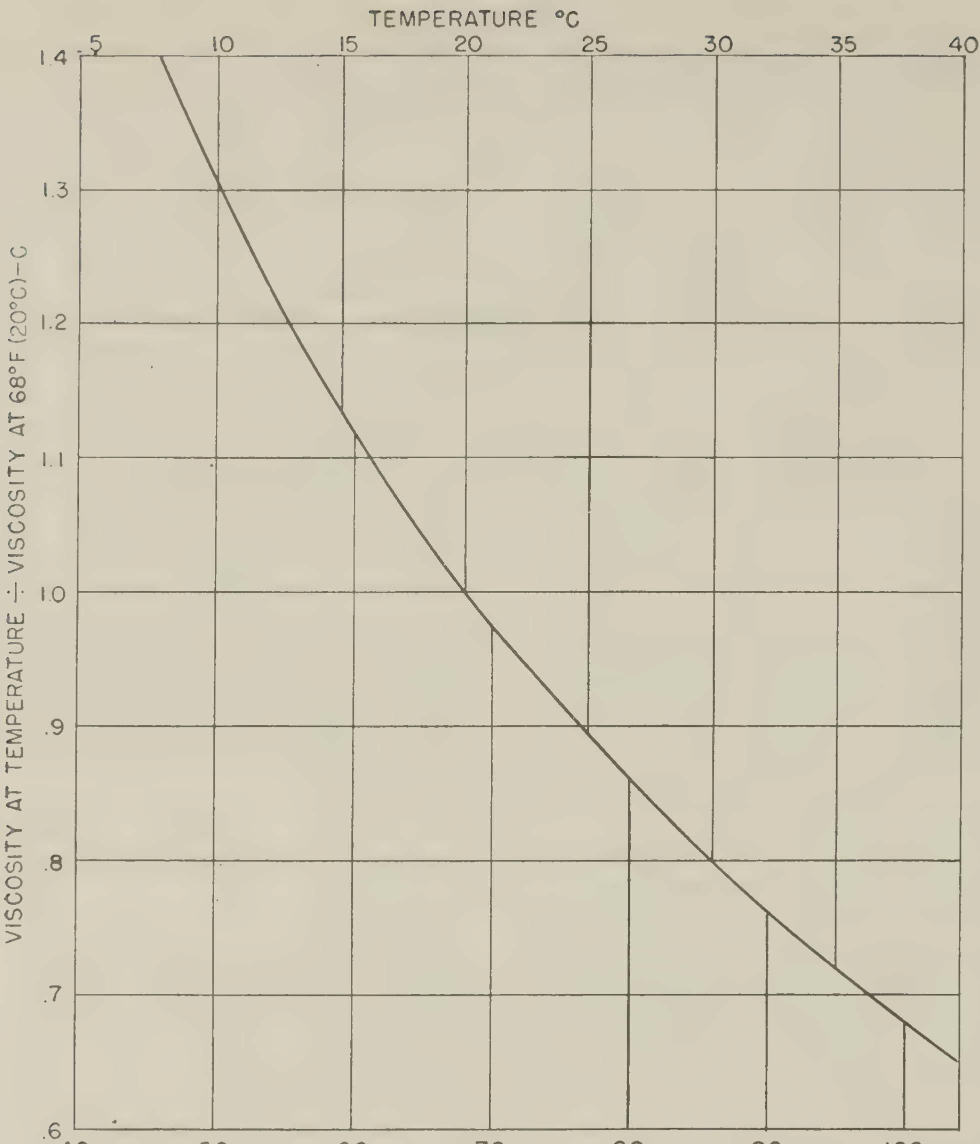


Figure 2.—Effect of temperature of water on the viscosity ratio.

$\Sigma d/\Sigma(d/k)$. If k_2 is large enough, the effect of the support is negligible. For instance, if $k_2=100k_1(d_2/d_1)$, neglect of the support introduces an error of only 1 percent.

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 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, $=0.00115/t$ inches per minute, or $0.138/t$ feet 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

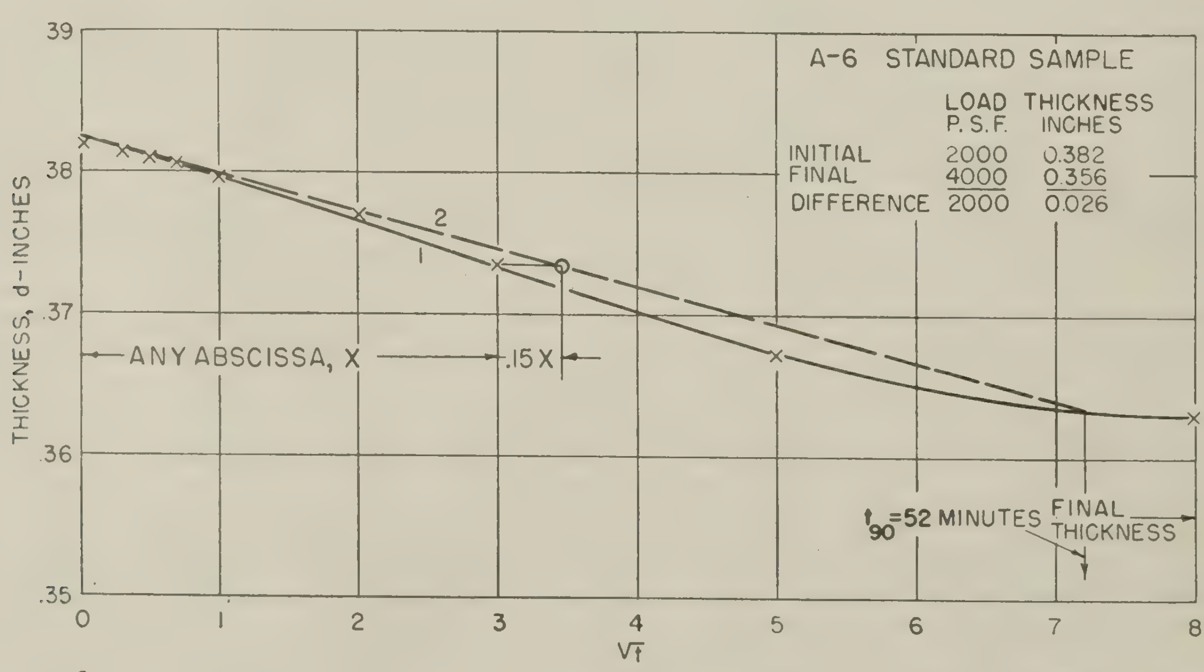


Figure 3.—Thickness change with time.

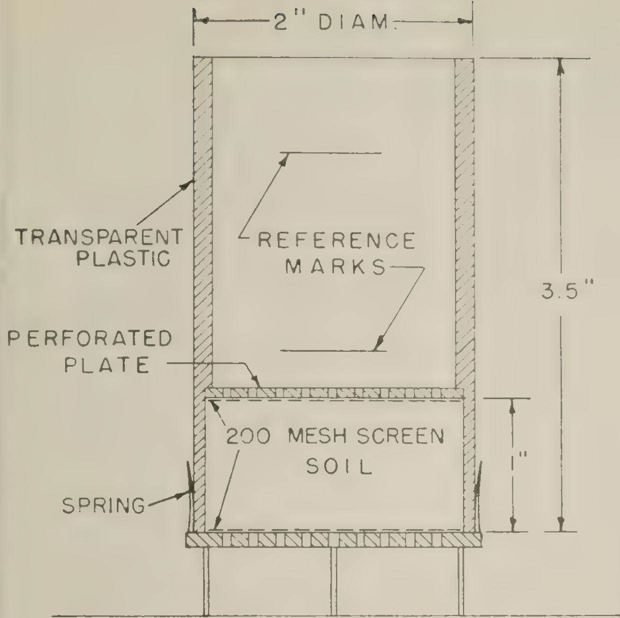


Figure 4.—Permeameter for granular soils.

over the surface of the soil. Thus, a soil with $k=1$ foot per day could transmit vertically downward a 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 inches of water.

Since the permeability depends upon the viscosity of the water, which is a function of temperature, the calculated permeability at an arbitrary temperature (68° F.= 20° C.) is often reported as $k_{68}=Ck$, where C is the viscosity of water at the test temperature divided by its viscosity at 68° F. Values of C are plotted in figure 2. Ac-

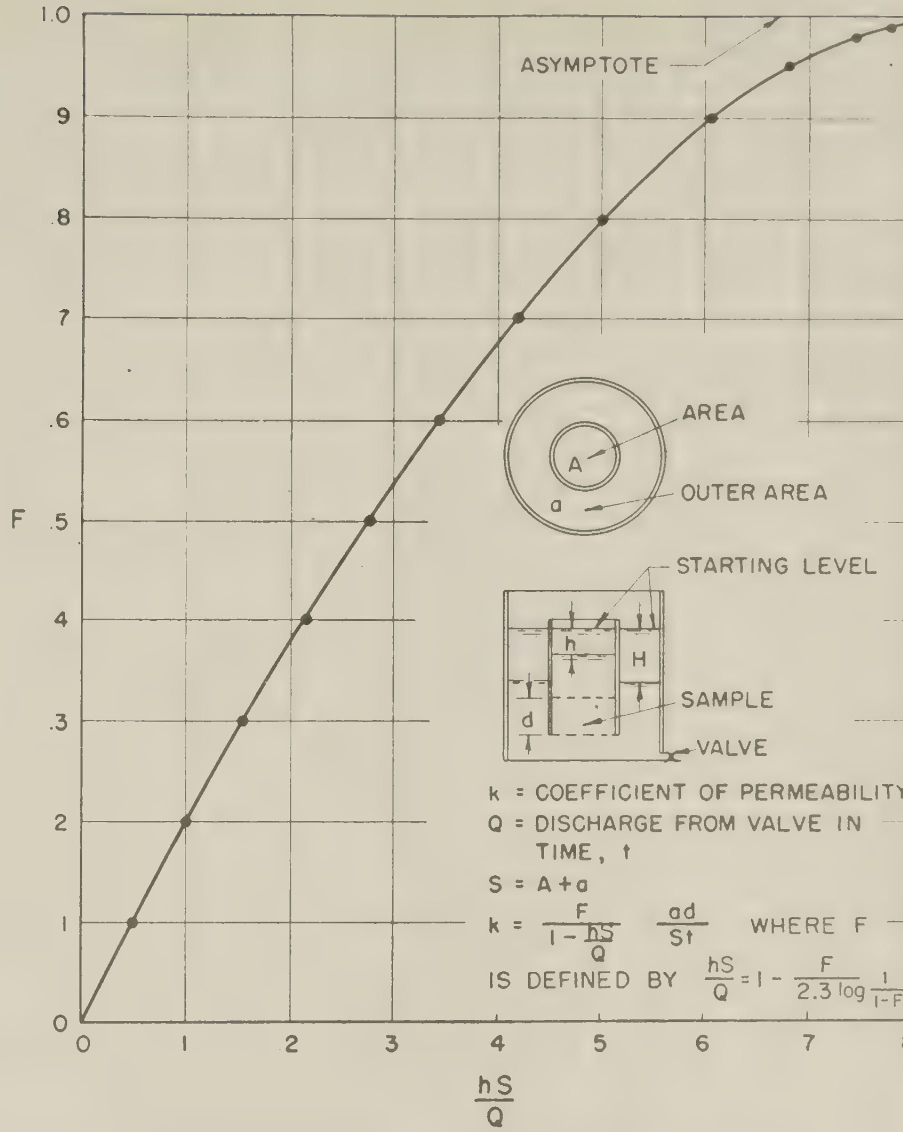


Figure 6.—Permeability from drainage-lag device.

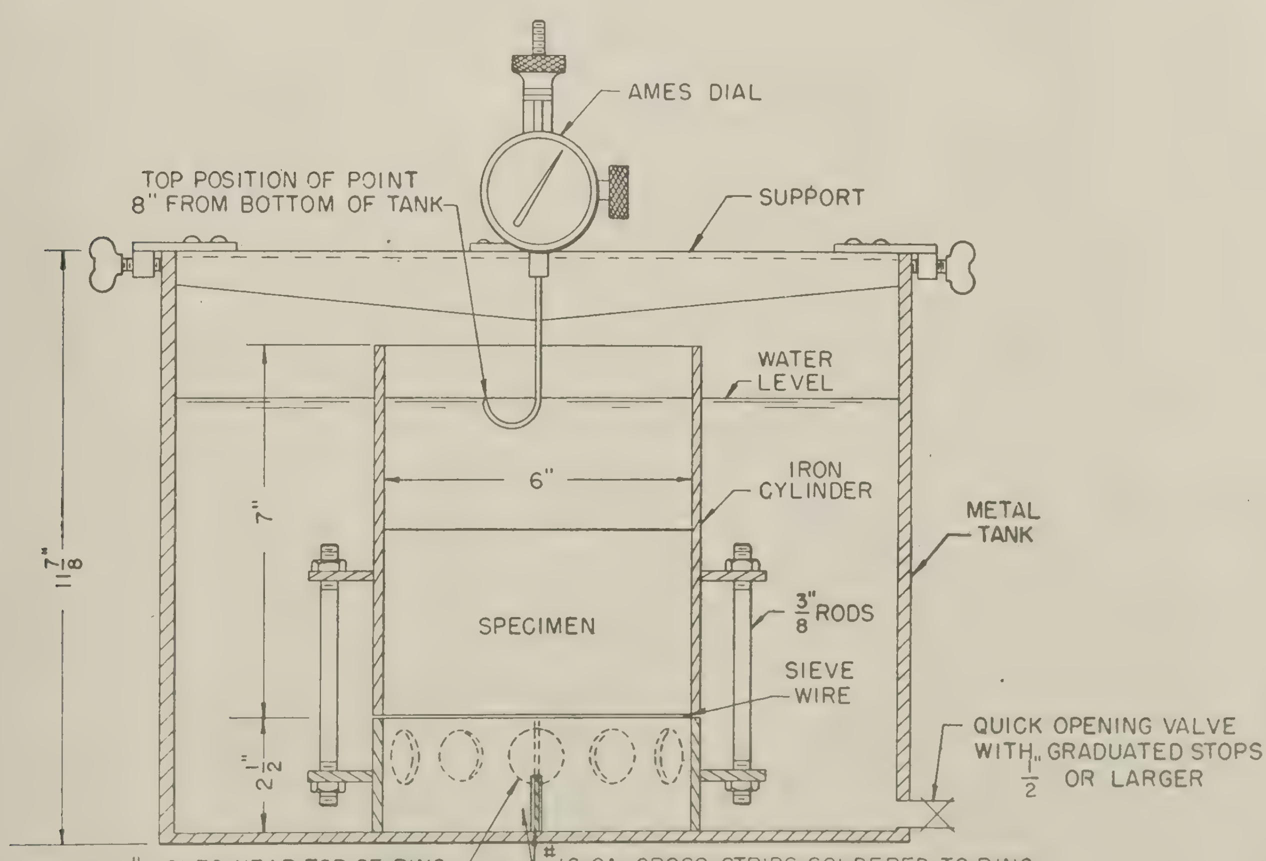


Figure 5.—Drainage-lag permeameter.

tually, 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 permeating 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:

$$k=132\frac{d\Delta d}{t_{90}\Delta p}\text{ feet per day} \quad (4)$$

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.

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:

$$k=132\frac{0.382\times0.026}{52\times2{,}000}=0.000013 \text{ foot per day.}$$

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.

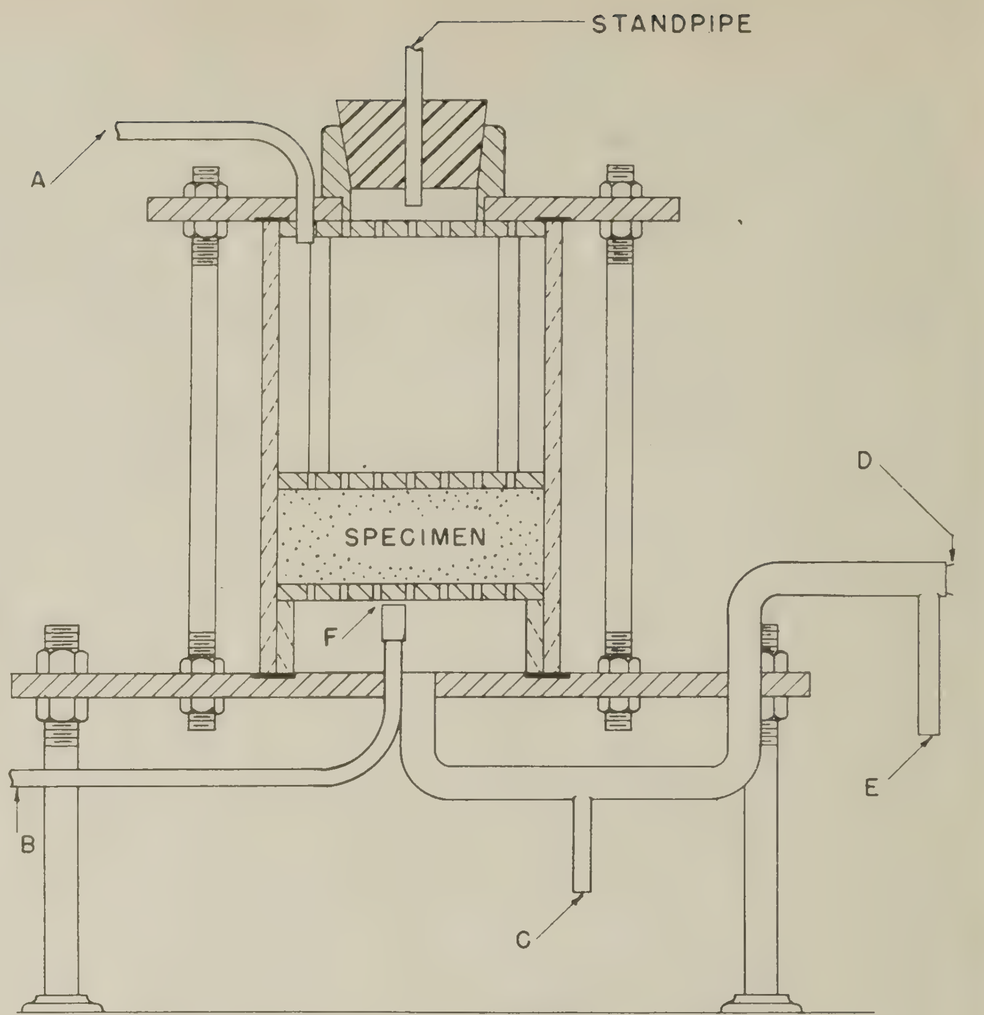


Figure 7.—Permeameter for saturation of sample under vacuum.

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 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. The effective head is a variable $(H\text{-}h)$ 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

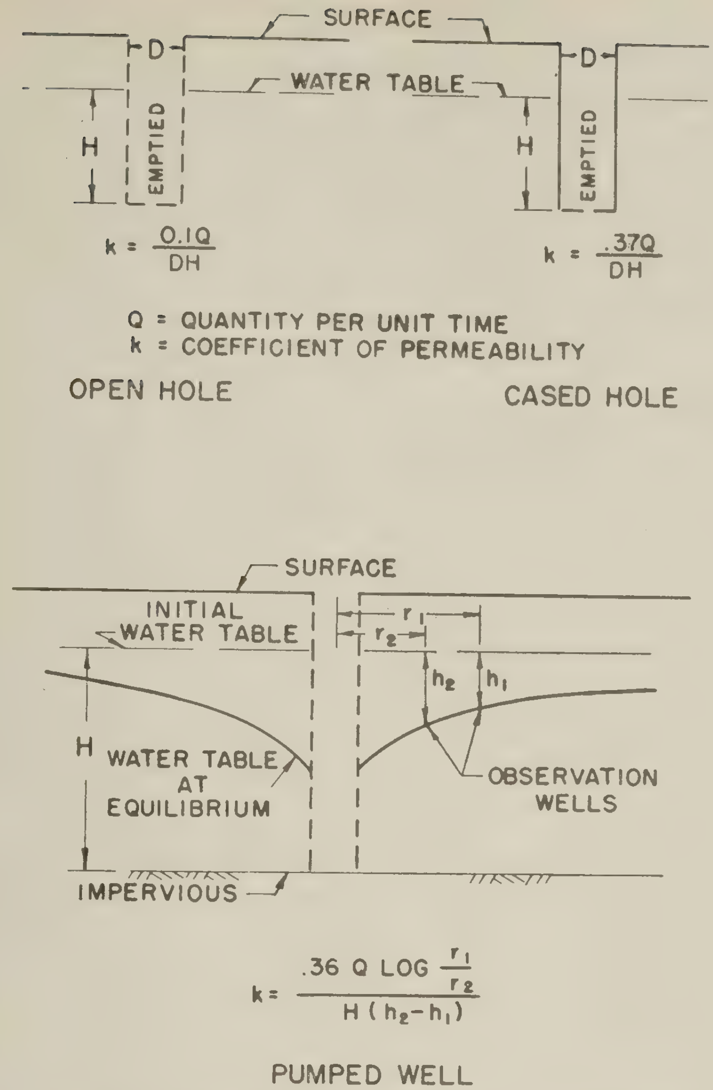


Figure 8.—Determination of coefficient of permeability in the field, below water table.

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*).[1]

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 in table 1. The soil classification is that published by the Bureau of Public Roads in 1942.[2] 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 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 square foot was 106 pounds per cubic foot, 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.

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*	*Lb./cu. ft.*	*Ft./day*	*Lb./cu. ft.*
				1	0.00045	114	0.0048	117
A-1	23	8	26	2	.00036	115	.0020	118
				4	.00024	117	.0017	118
				1	.00048	103	.0047	99
A-2	28	11	40	2	.00028	106	.0016	102
				4	.00018	109	.0009	108
A-3	NP	NP	0	(1)	200	106	226	101
				1	.00030	94	.00075	90
A-4	33	12	99	2	....	97	.00060	94
				4	.00022	100	.00024	99
				1	.021	84	.37	78
A-5	35	6	35	2	.015	86	.25	80
				4	.012	90	.17	84
				1	.000035	75	.00011	89
A-6	72	45	86	2	(2).000019	80	.00011	90
				4	(2).000009	86	.00010	92
				1	.00051	77	.026	78
A-7	67	34	71	2	.00023	81	.0084	81
				4	.00020	86	.0012	83
				1	.0039	43	.041	47
A-8	78	8	38	2	.0010	46	.018	48
				4	.00046	49	.010	50

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

[1]Italic numbers in parentheses identify the references in the appendix, page 268.
[2]*Classification of soils and control procedures used in construction of embankments*, by Harold Allen. PUBLIC ROADS, vol. 22, No. 12, Feb. 1942.

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

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 pounds per square inch and then heated to 140° F. for 24 hours 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 foot 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 feet per day.

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

Permeability of Graded Aggregates

The material graded from the ¾-inch 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-inch and No. 200 sieves. The standard A.A.S.H.O. compaction test was made on each mixture except that the material retained on the No. 4 sieve was not removed

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

Table 6.—*Gradation of permeability samples for base-course mixtures passing ¾-inch 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

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 110° C., washed on a No. 200 sieve, and the sieve analysis determined. The other sample was air-dried, the sieve analysis determined by the standard A.A.S.H.O. method, and the fraction passing the No. 40 sieve was tested according to standard A.A.S.H.O. 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 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.

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: Maximum density	Compaction test: Optimum moisture	Molding moisture	Permeability coefficient (68° F.): Test 1	Permeability coefficient (68° F.): Test 2	Permeability coefficient (68° F.): Average
			Lb./cu. ft.	*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

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 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 A.A.S.H.O. 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 interpolated 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

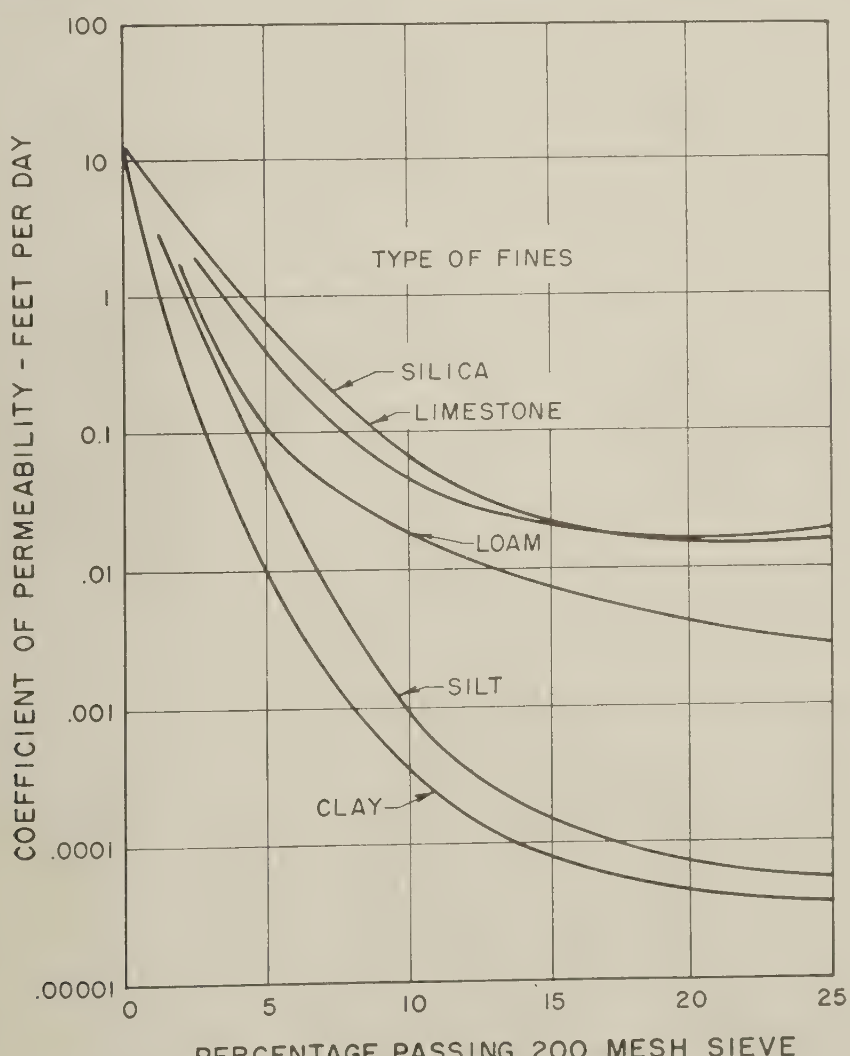


Figure 9.—Effect of fines on permeability of graded aggregate.

Table 8.—*Gradation 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

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 (68° F.)		
	Density	Moisture	Test 1	Test 2	Average
	Lb./cu. ft.	*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
Limestone 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	....	....

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 right-hand 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.

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 18° F. (10° C.).

At a given height, different materials may be at equilibrium with quite different amounts of water: For instance, at a height of 2 feet, 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 foot may be at equilibrium with moisture contents from 12 to 20 percent at different times depending upon the previous moisture variations.

Specific Yield

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

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

where

m_o=moisture content before drainage, in percent.

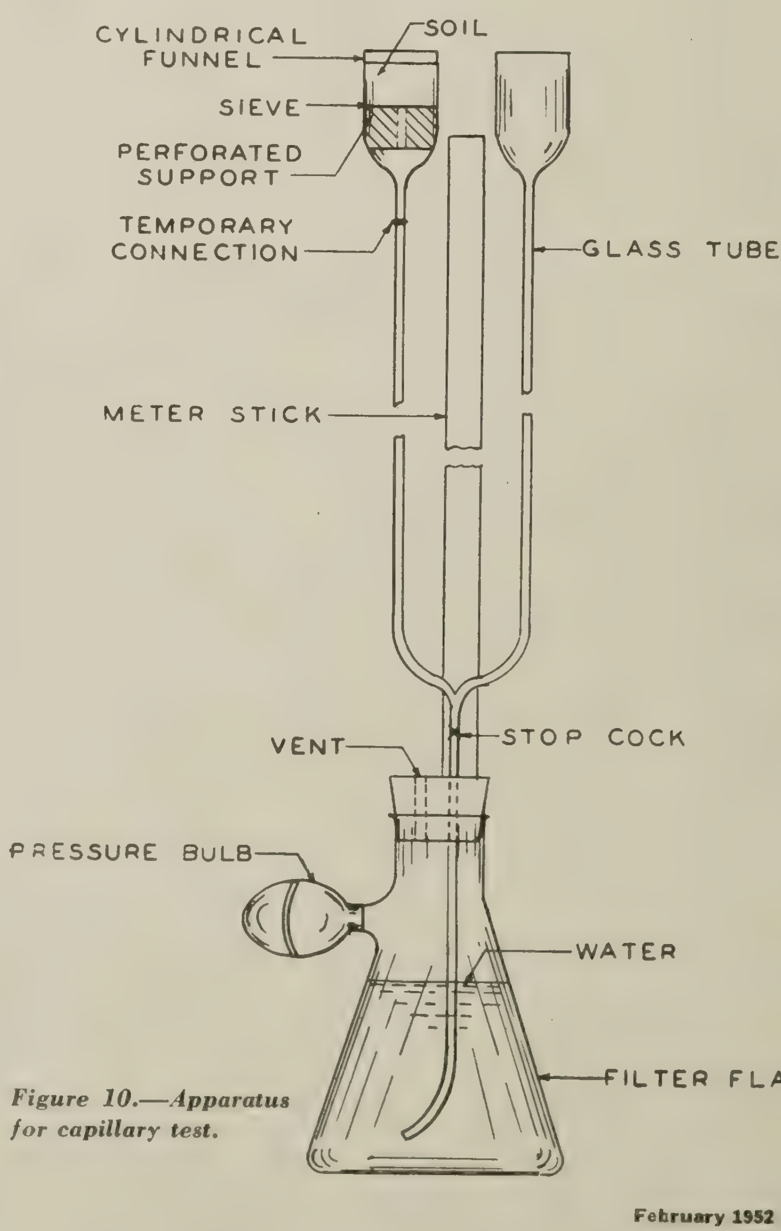


Figure 10.—Apparatus for capillary test.

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 capilary fringe of height H_1 above the water able when the water table is lowered a lepth d, m may be taken as the average noisture held against heights between H_1 ınd H_1+d.

This is illustrated in figure 12. The nitial condition shows that the soil to be lrained had a moisture content of m_0 and lepth d with a capillary fringe above the vater table of depth H_1. The final condiion shows the water table lowered a depth l with a capillary fringe of depth H_1+d. The capillary fringe of depth H_1 is common o both conditions so that the moisture lrained is from m_0 for the initial condition o the average moisture m in the capillary ringe over the distance d from H_1 to H_1 $+d$ above the water table.

For example, consider the sand-clay curve n figure 11, assuming $H_1=1$ and $d=3$. Iere $m_0=41$ and the average moisture conent from $H_1=1$ to $H_1+d=4$ is $m=29$. ıssuming $w_0=90$, then $y=0.17$, from equaion 5.

To determine the specific yield of graded and with various additives passing the No. 00 sieve, the materials represented in table were tamped in lucite tubes of 1¾-inch nside diameter to a depth of 47 inches at ptimum moisture and maximum A.A.S.H.O. lensity. The samples were supported by piece of 200-mesh screen wire held by a erforated rubber stopper. In some cases, ome water came out of both the top and ottom of the tubes after compaction, aparently due to excess pressure built up n the air trapped in the wet soil. After everal days a head of water was applied t the bottom of the tubes, creating an upvard gradient until water flowed out the op. The purpose was to saturate the amples but, as shown in the first four lines f table 10, the air was not readily displaced and saturation could not be acomplished. Thus, for the material with -percent limestone added, the compaction noisture was 7.0 percent. This increased o 10.1 percent before drainage, but 13.4 ercent was required for saturation.

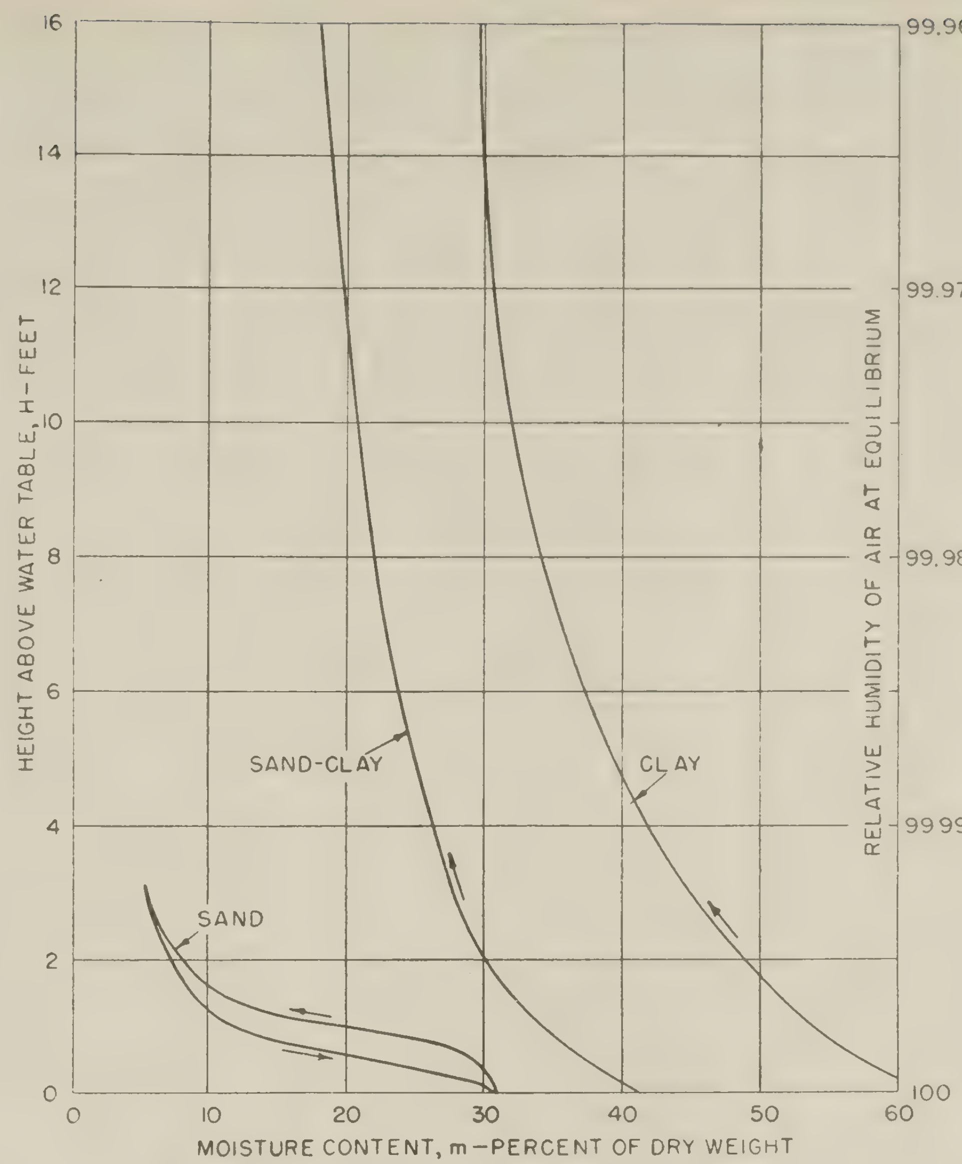


Figure 11.—Moisture-height relations for various soils.

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-inch increments, starting from the top, and the moisture content of each increment determined. The results are shown in table 10. The moisture contents for

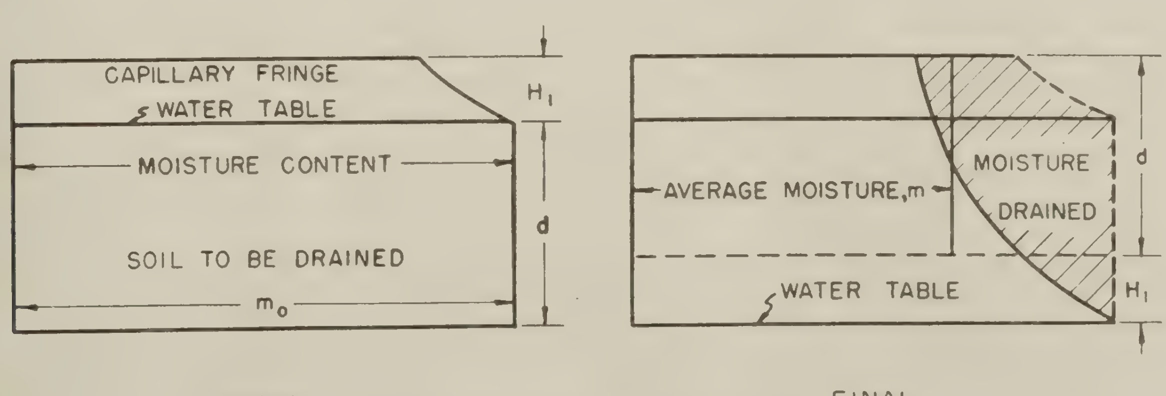


Figure 12.—Specific yield including water from capillary fringe.

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 sieves [1]									
	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./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 [2] after drainage, percent:										
46 inches	3.9	6.1	5.1	5.6	5.6	7.7	6.2	6.4	9.5	8.7
44 inches	3.9	6.2	5.2	6.0	5.6	7.8	6.2	6.4	9.0	8.0
42 inches	4.1	6.0	5.2	5.8	5.7	7.9	6.6	6.4	8.6	9.0
40 inches	4.2	6.4	5.4	5.9	6.0	8.2	6.7	6.8	8.7	8.7
38 inches	4.4	6.4	5.6	6.7	6.1	8.0	6.9	6.7	8.7	8.6
36 inches	4.4	6.6	5.4	6.0	6.3	8.6	7.0	6.9	8.5	8.1
34 inches	4.6	6.7	5.4	6.7	6.5	9.2	7.0	7.0	8.8	8.0
32 inches	4.8	6.8	5.6	6.0	6.6	9.5	7.0	7.1	9.0	8.3
30 inches	5.3	6.7	5.5	6.0	7.0	9.9	7.1	7.7	9.1	8.8
28 inches	5.4	6.9	5.7	6.0	7.1	9.8	7.2	8.2	9.3	8.2
26 inches	5.9	7.1	5.7	6.0	7.4	9.9	7.0	9.0	8.4	9.4
24 inches	6.1	7.3	5.8	6.0	7.6	10.1	7.4	9.0	8.6	9.5
22 inches	6.5	7.6	6.0	6.1	7.7	9.7	7.7	9.0	8.2	9.8
20 inches	7.1	8.0	6.0	6.6	7.8	9.7	8,1	8.9	8.8	9.3
18 inches	7.8	7.9	6.1	6.1	8.2	10.0	8.1	8.8	8.8	10.0
16 inches	8.2	8.2	6.2	7.6	8.6	9.7	8.1	8.7	8.6	9.8
14 inches	8.3	7.7	6.3	7.1	9.1	9.3	8.5	8.4	8.5	9.9
12 inches	8.1	7.9	6.5	7.1	9.0	8.6	9.0	8.4	8.8	10.0
10 inches	8.1	7.8	7.2	7.1	8.8	8.6	8.8	8.1	8.9	10.6
8 inches	7.9	8.0	7.2	6.7	8.9	8.6	8.9	8.0	9.0	9.8
6 inches	8.2	8.2	7.8	7.0	9.2	8.5	9.2	7.4	9.0	10.0
4 inches	8.5	8.8	8.5	7.3	9.5	9.1	8.6	7.4	9.2	8.9
2 inches	9.1	9.2	9.5	8.2	10.1	9.7	8.8	8.4	9.5	9.1
½ inch	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

[1] Physical characteristics of admixtures are shown in table 7.
[2] Center of 2-inch increments except bottom inch.

various heights above the bottom of the tube for the specimens containing 5-percent limestone dust and 5-percent Keyport silt loam are shown graphically in figure 13. Using equation 5, the specific yield was calculated for lowering the water table from the surface to various depths, and the values are shown graphically 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-inch 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 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, 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 fo k_u in equation 6, and integrating, gives:

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

For a layer in contact with the water tabl ($z_o=H_o=0$), the maximum capillary flo 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 c flow is:

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

With the values of a and b noted abov this equation gives the values plotted i figure 15 which shows that, while the cla can lift 1 inch of water per year to th greatest height, the silt gives the max mum height for larger rates comparab to those required for appreciable fro heave. Rates of flow due to capillari may be much greater than those due gravity alone. For instance, while the cla (fig. 15) can lift 100 inches of water p year 0.6 foot, the maximum rate of gravi flow (unit hydraulic gradient) is only 0.0 foot per day (k_u for H=0 which equals or 22 inches per year. Similar effects ha been noted with portland cement concre for which the water transmitted by capi larity is much greater than that forc through a sound sample by ordinary pre sures.

Effect of Layers

The effect of a layer of sand on the c pillary rise of water is shown in figure 1 (Figs. 16 and 17 show soil cross sectio and corresponding moisture content curves With an impervious surface, so that the is no flow through the surface, as show on the left side of figure 16, each materi attains the moisture it would hold at given height if it were continuous to t water table—that is, the sand does not a fect the moisture in the silt at static equ librium. If water evaporates continuous from the surface, as at the right in figu 16, the soil will dry out enough to esta lish a tension gradient sufficient to mai tain the flow required for continuous e eration. The sand causes more drying the top layer than would occur without because the unsaturated permeability the sand at appreciable height above t

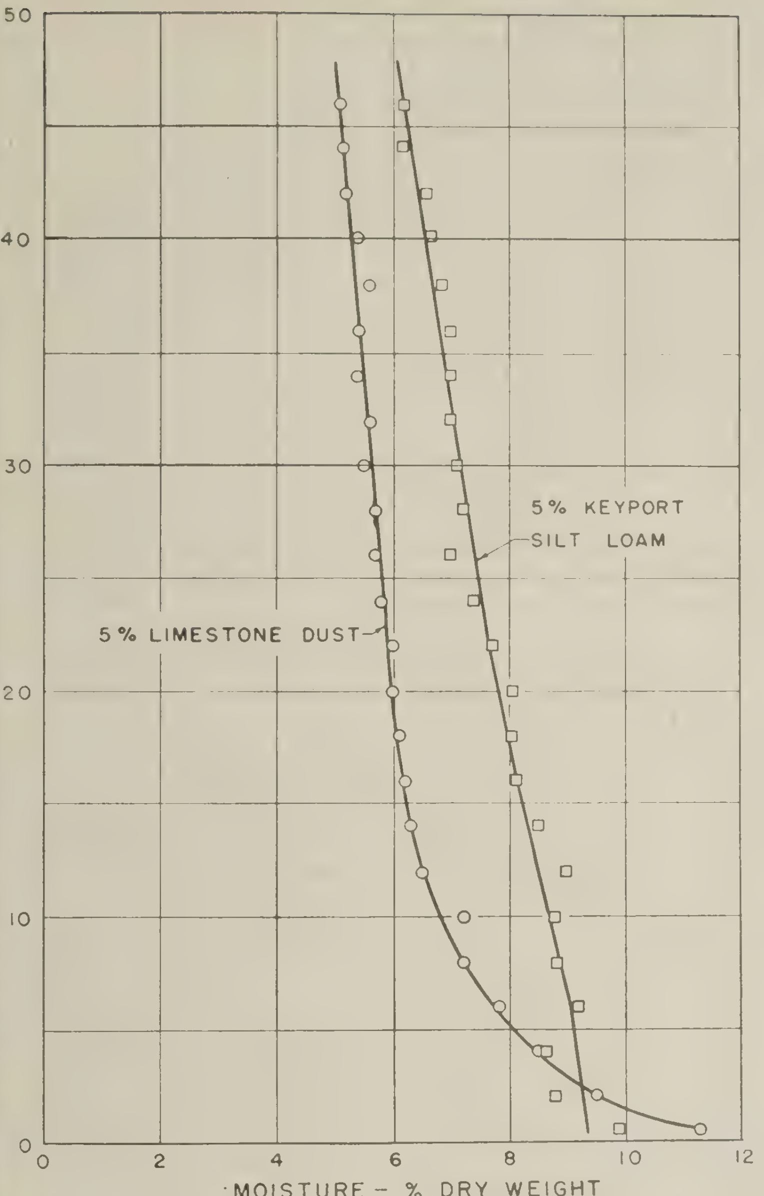


Figure 13.—Capillary retention in drained columns of soil.

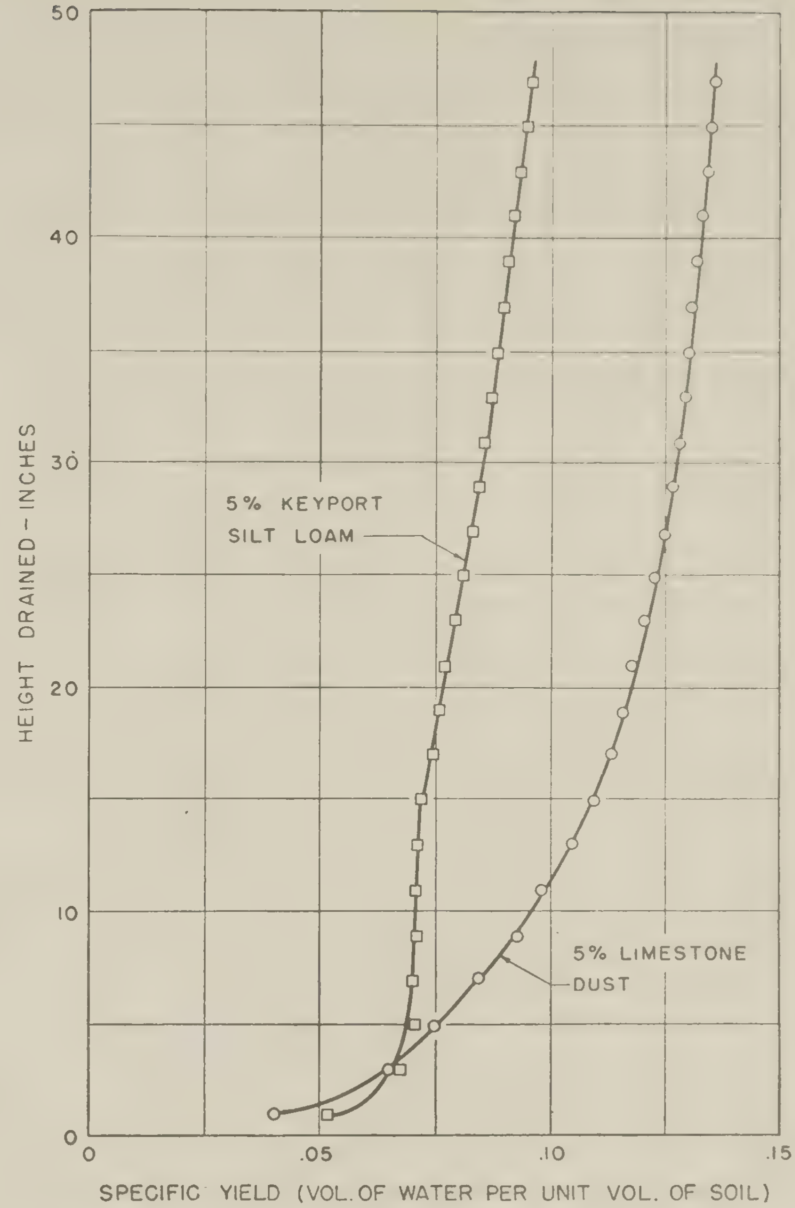


Figure 14.—Specific yield for various thicknesses, assuming initial saturation.

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

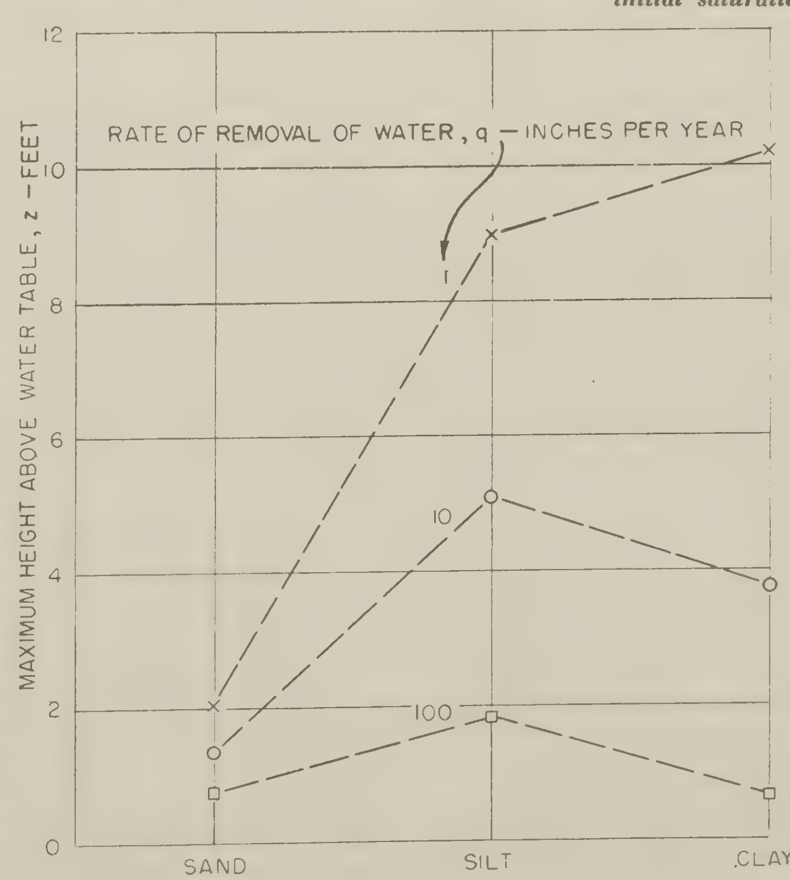


Figure 15.—Calculated rates of capillary flow.

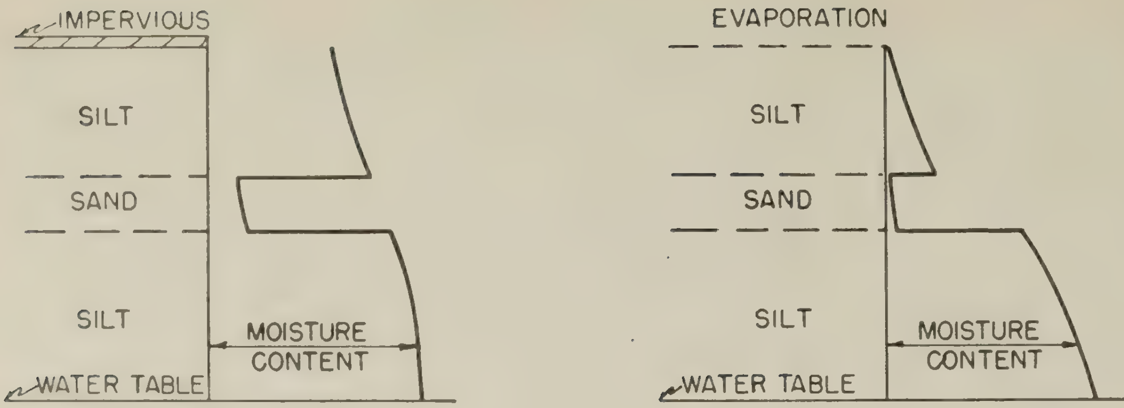


Figure 16 (left).—*Capillary rise of water.*

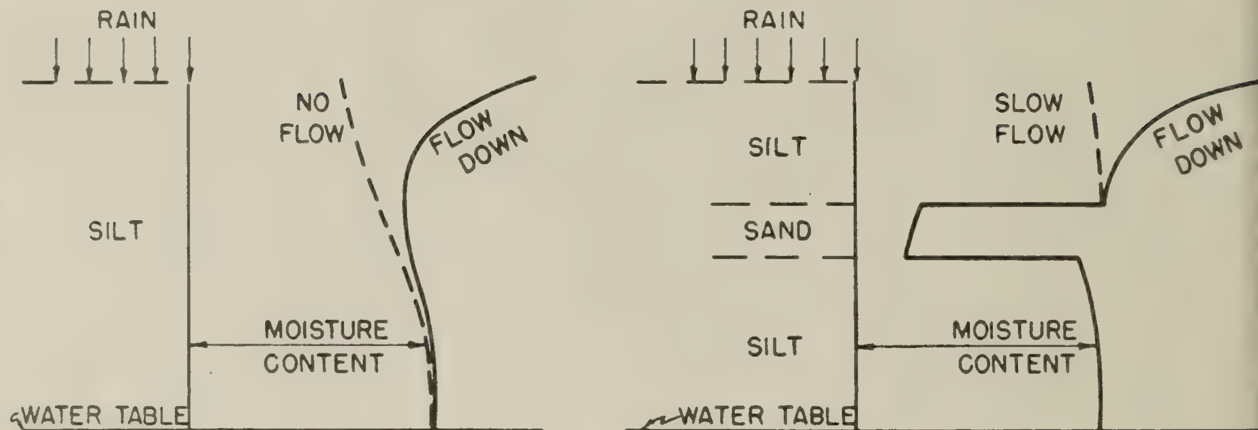


Figure 17 (right).—*Downward capillary flow.*

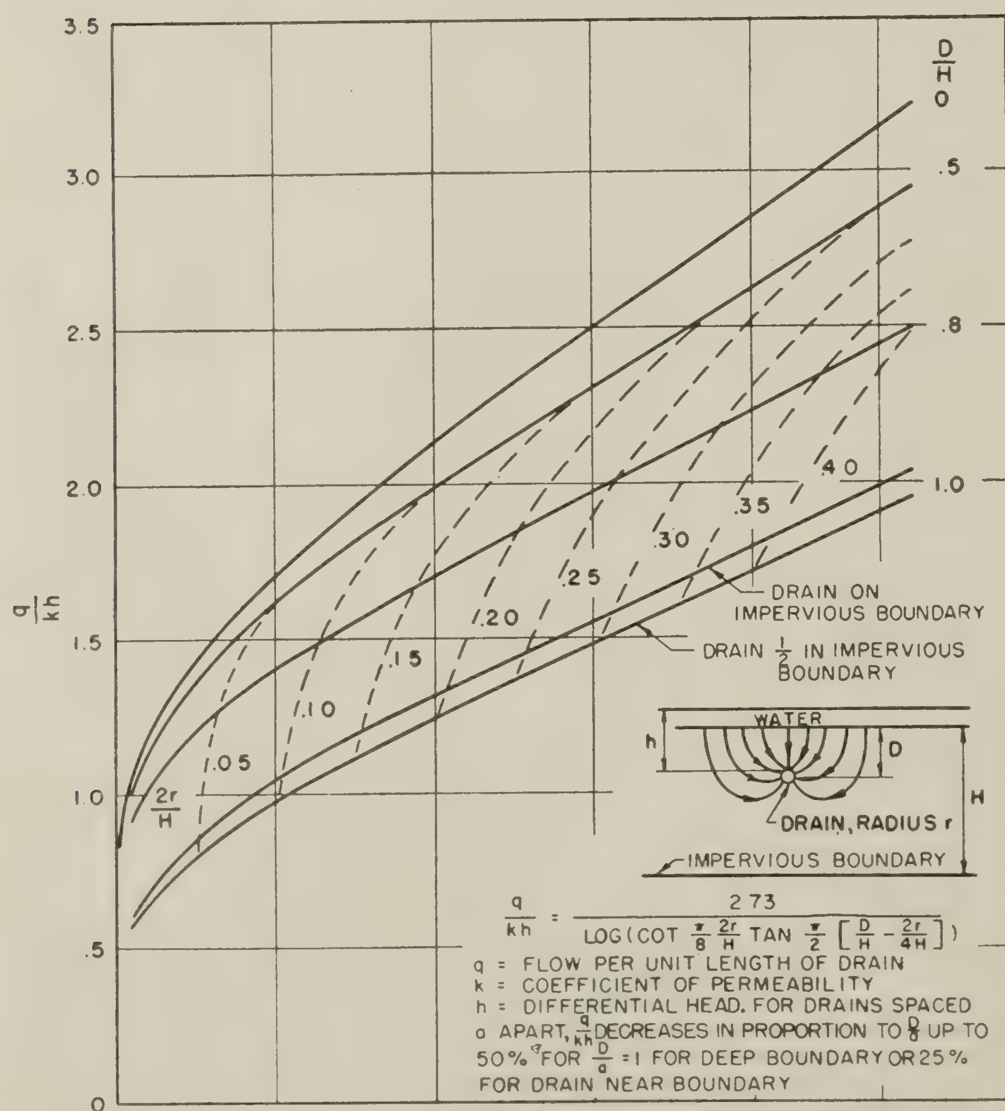


Figure 18.—Flow into buried horizontal drain from flooded surface

low in a subgrade, a base on the subgrad may drain better than one placed on highly porous subbase.

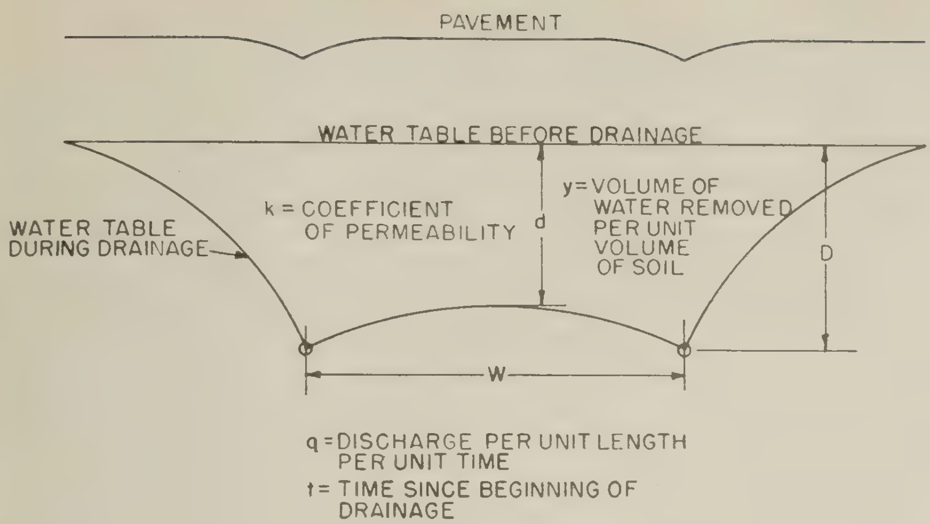
Other Influences

Temperature difference in the soil cause flow of water since, for equal initial mois ture content, water tends to move from warmer to a cooler area. Another forc which causes water to flow is the osmoti pressure such as is caused by difference in concentration of salts at different lo cations in the soil profile. Flow of so water may also be caused by an electric potential. Thus, if two electrodes ar placed in a soil and a direct current passe between them, water will flow from th vicinity of the positive electrode to th negative electrode, which could be con structed as a drain. The high cost of th reported field applications which have bee made may possibly be overcome by a com prehensive study of this method.

Water may also move through a soil a a vapor. If the air moves as a body, con siderable water may be transferred b convection. If the air is still, the vapo may move by diffusion but this is very slow Since the soil air is generally so near saturated, a small decrease in temperatu will cause condensation. In some arid re gions water has accumulated under pave ments, apparently from condensation a sociated with the rapid cooling of the su face due to radiation under clear skies.

HORIZONTAL PIPE DRAINS

Figure 18 shows the flow of water in buried horizontal drains from a floode surface, as derived by Kirkham (7). Th



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.

ater on the surface could be a film due o rain or water in a permeable base on a uch less permeable subgrade. The type f drain is not considered; it is assumed hat there is negligible resistance to water ntering the drain.

For example, for 6-inch drains, in lines paced a=10 feet apart, resting on an im-ervious boundary at a depth H=5 feet: r/D=0.5/4.75=0.105, and figure 18 gives $/kh$=1.05. For a drain near the imper-ious boundary, this is reduced by D/a imes 25 percent of its value. Thus, the djusted q/kh=1.05−(4.75/10)×0.25×1.05 =0.93. For h=4.5, q=0.93×4.5k=4.2k. For =0.1 foot per day, q=0.42 cubic foot per day for each foot of length. This is equivalent to (0.42/10×1)×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. Suppose the time and rate of flow are desired when d/D reaches 0.79. Then d in figure 19 is 0.79×3=2.4 feet; tkD/yW^2=0.1, so that t=(0.1×0.1×30×30)/(0.25×3)=12 days; and q/kD=0.25, giving q=0.25×0.25×3=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:

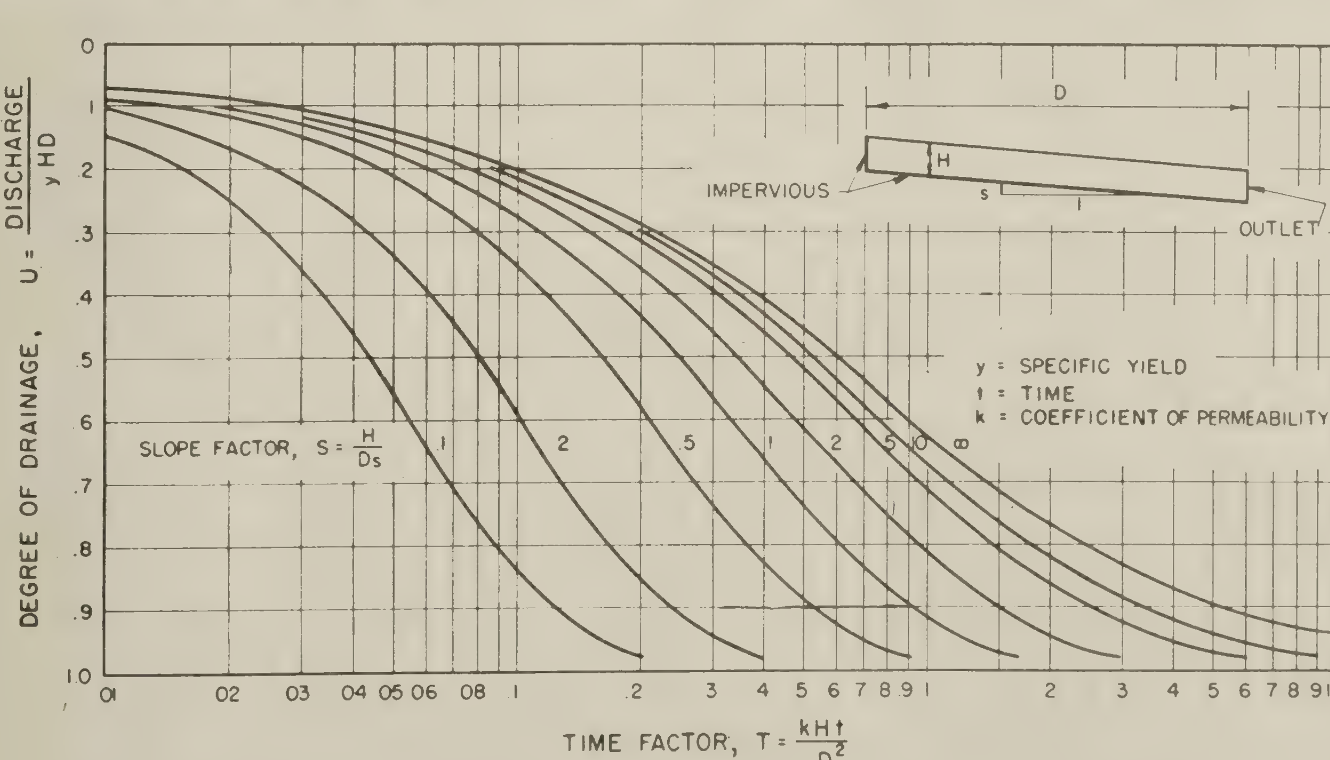


Figure 20.—Rate of drainage of flooded base course.

$T = US - 0.48S^2 \log (1 + 4.8U/S) \ldots (10a)$
for $0 \leqq U \leqq 0.5$, and

$T = 0.5S - 0.48S^2 \log (1 + 2.4/S) + 1.15S \log (S - US + 1.2)/(1 - U)(S + 2.4) \ldots (10b)$
for $0.5 \leqq U \leqq 1$

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:

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

Curves derived by substituting values in formulas 10 are plotted in figure 20. For U = 50 percent, this relation may be closely approximated by:

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

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) $= 0.5 + 20 \times 0.01 = 0.7$ 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 $t = (0.05/0.51) \times (0.44 \times 20^2)/(0.74 \times 0.5 + 20 \times 0.01) = 30$ 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:

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

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

This corresponds to the rate of flow fo[r] 42-percent drainage starting from a satu[-] rated condition. Thus, roughly, for infil[-] tration through a joint the base would be[-] come only 58-percent saturated if drainag[e] 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 = $[1 \times 0.5(0.4 + 0.25)]/20 = 0.016$ cubic foot pe[r] day as the maximum quantity that coul[d] be transmitted continuously from a joint o[r] crack, any available excess being forced t[o] run off. Over an area of 20×1 foot, thi[s] quantity of water could be transmitted ver[-] tically into a drained subgrade if its per[-] meability were as much as 0.0008 foot pe[r] day. If the base had a k value of 100 fee[t] per day, it could transmit 1.6 cubic feet pe[r] day which would require a subgrade per[-] meability of 0.08 foot per day for continu[-] ous infiltration. While a dense-grade[d] base of low permeability will not transmi[t] water as fast as an open-graded base, i[t] will not have as much total capacity fo[r] water, must lose much less water for [a] given percentage of drainage from a floode[d] condition, and may make much less wate[r] available to the subgrade by its lower in[-] filtration capacity.

WATER AND STRENGTH

The presence of water in a base materia[l] may decrease its strength in several ways[.] It reduces the cohesion by lowering th[e] capillary forces; it reduces the frictio[n] by reducing the effective weight of th[e] material below the water table; and, fo[r] quickly applied loads, it may reduce th[e] strength by the development of pore pres[-] sure.

Tests made on sands by other investiga[-] tors (*11*) showed the bearing capacity t[o] be decreased more than 50 percent du[e] to complete submergence as compared t[o] dry sand. Capillary saturation gave some[-] what less reduction. Under dynamic load[s] even greater loss in strength was obtained[.] The effects of wetting were especially no[-] ticeable for low initial densities of th[e] sand.

Rise of water table in the base also a[f-] fects the strength of subgrade by reducin[g] the effective pressure on the subgrad[e.] For example, if a 12-inch base and surfac[e] course weighing 140 pounds per cubic foo[t] is submerged, the effective weight is re[-] duced to 140—62.4=77.6 pounds per cubi[c] foot and the effective pressure on the sub[-] grade is reduced from 140 to 77.6 pound[s] per square foot. According to figure 2[…] which shows the strength of a clay at equ[i-] librium under various surcharges, the com[-] pressive strength of the subgrade may b[e] reduced from 1.4 to 1.0 kips per squar[e] foot by the submergence of the base.

The strength of granular materials unde[r] quickly applied loads is greatly affected b[y] ensity, especially if saturated. This is ecause the strength depends on the ef- ective stress, which is the total stress iinus stress on water or pore pressure, nd the pore pressure depends on the den- ity. Thus, if a loose saturated granular iaterial is distorted by shear stresses, the endency to become denser causes pore pres- ure (as shown in the upper part of figure 2), which reduces the effective normal tresses and thereby reduces the strength. 'n the other hand, a dense material tends) expand when sheared (as shown in the ›wer part of figure 22) so that no pore ressure is developed and the total pressure ; effective in developing shearing resis- ance through internal friction.

The fact that some bases appear stable 'hen saturated may be due to the fact that iey are dense enough so that they must ex- and to shear even after a loss of density ue to freezing or other causes.

For materials near saturation, freezing iay cause a decrease in density due to the xpansion of water in freezing, even with- ut ice lenses. Thus, 20-percent water by olume upon freezing increases to 22 per- ent, causing a 2-percent decrease in den- ity. It has been suggested by C. H. Mc- 'onald (*12*) that materials which do not iove during compaction (displace verti- ally upward at high moisture contents) 'ill be stable when properly compacted re- ardless of wet and freezing conditions.

While high permeabilities may be obtained y using coarse aggregates, care must be iken to prevent a reduction in their per- ieability and stability by intrusion of ner soils to which they may be placed ad- icent. For instance, if two layers of ma- erial are used to make up the test sample i the apparatus shown in figure 23, and repetitive load similar to traffic loading ; applied by means of the motor and cam, ie finer material will be intruded into the ores of the coarser material if the differ- nce in size is too great. Thus, many maca- am roads placed on clay have failed when ie clay became wet and soft, as indicated i the sketches on the left side of figure 24. Vell-graded aggregate or a fine aggregate ubbase, as shown on the right side of fig- re 24, will prevent intrusion.

Use of Filters

Subsurface drainage of soil in cut slopes nd under pavements is often accomplished y the use of drain tile or perforated pipe laced in a trench and covered with a gran- lar material. The granular material is ommonly designated as the filter. If the oids of the filter material are very much irger than the finer grains of the soil to e drained, the fine soil particles are likely) be washed into the pipe or into the in- erstices of the filter (left and center ketches, fig. 25), where they accumulate nd gradually obstruct the flow.

When the finer particles of the filter ma- erial at or near the plane of contact with

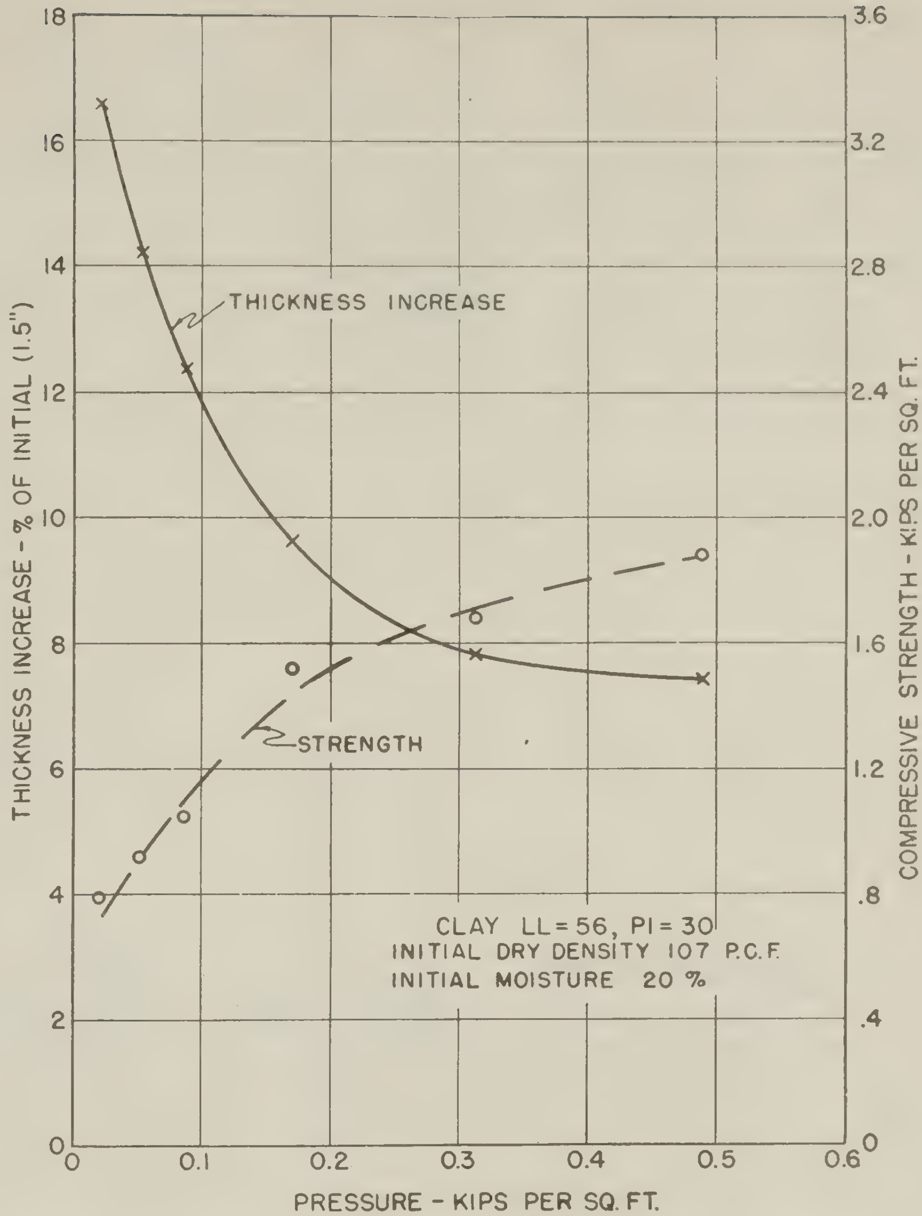


Figure 21.—Strength of clay immersed under various surcharges.

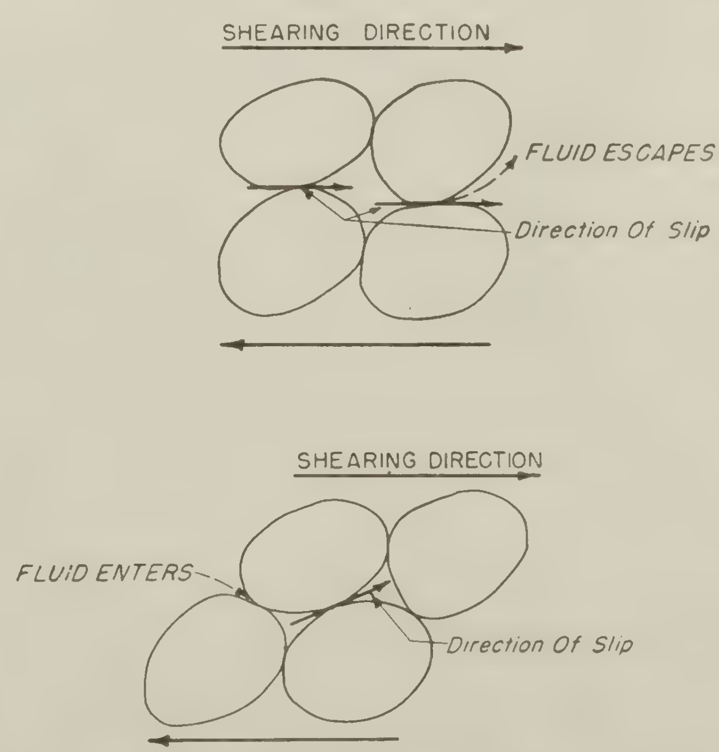


Figure 22.—Volume change with shearing.

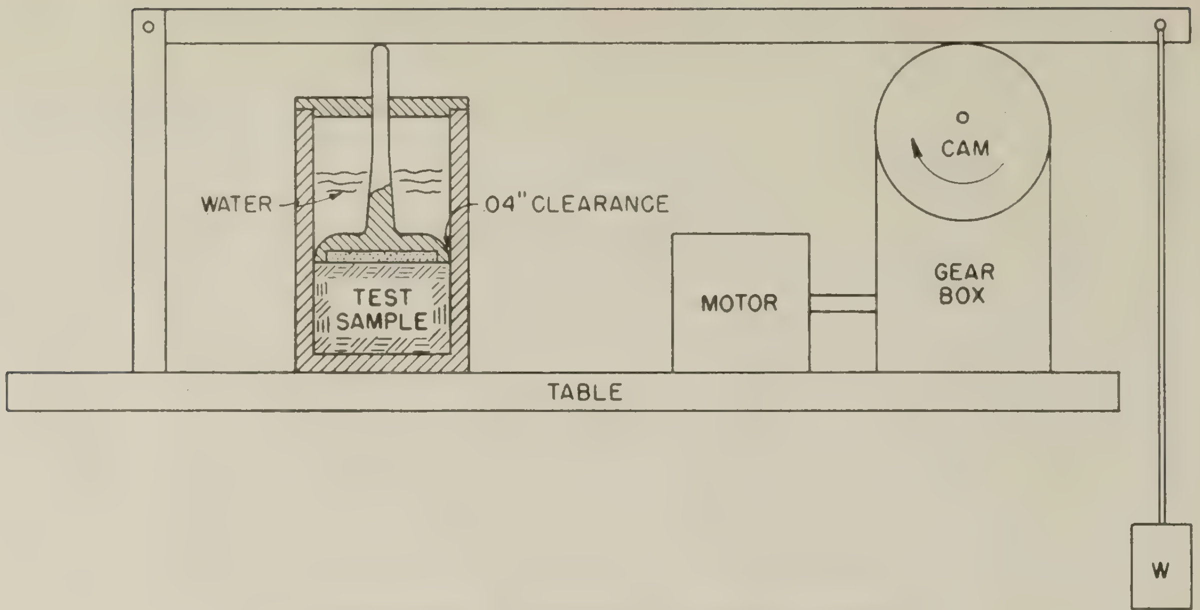


Figure 23.—Apparatus for repetitive loading.

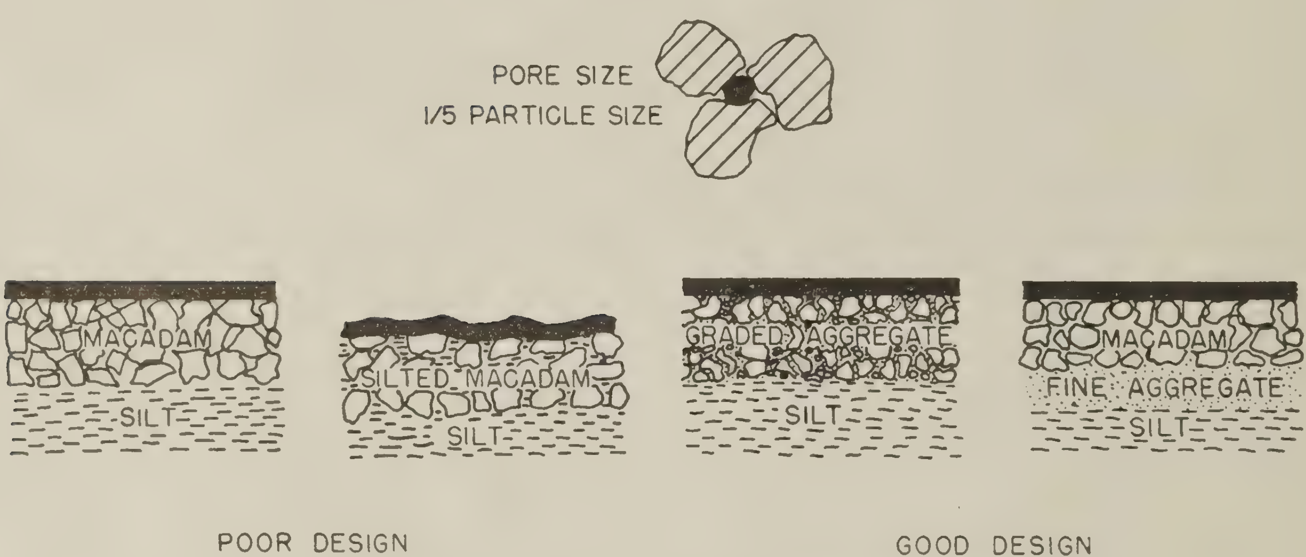


Figure 24.—Intrusion of silt into open aggregate.

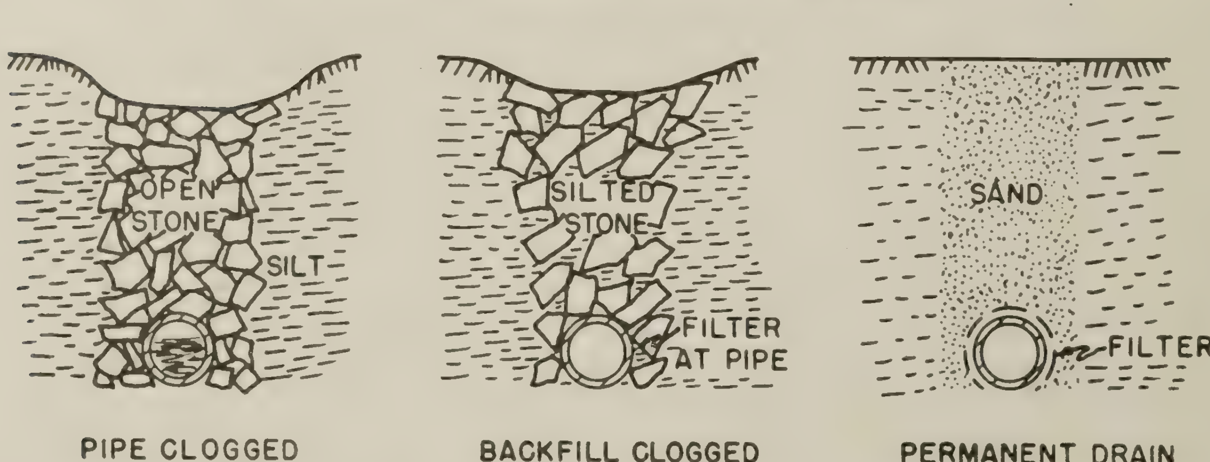


Figure 25.—Intrusion of silt into drains.

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 ra[tio] of 15-percent size of filter material to 8[5-] percent size of material to be drained [be] equal to or less than 5. For plastic cla[ys] with sand or silt partings (extremely th[in] seams) the 15-percent size of the filter—the size than which 15 percent by weig[ht] is finer—should be compared to the 85-p[ercent]

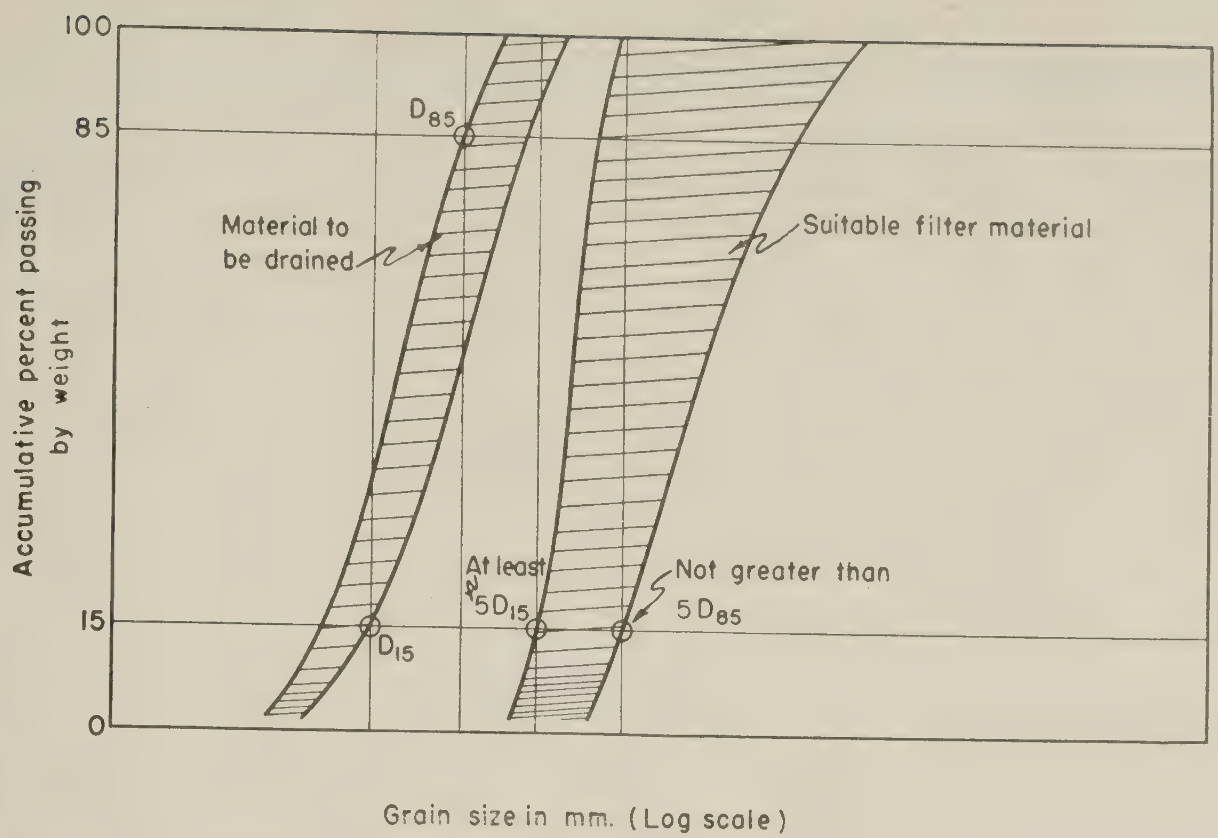


Figure 26.—Specification for grain size of material suitable for filter.

cent 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 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 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.

Table 12.—*Compaction of gravel mixtures under repetitive loading*

Material, and percentage passing No. 200 sieve	Dry density after—			Thickness reduction—static load to 10,000 repetitions
	Static load, 16 kips per sq. ft.	100 repetitions, 4 kips per sq. ft.	10,000 repetitions, 4 kips per sq. ft.	
	Lb./cu. ft.	*Lb./cu. ft.*	*Lb./cu. ft.*	*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	[1] 135	5.8
15 percent passing	130	131	[1] 132	3.7
25 percent passing	130	130	[1] 130	2.1

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

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF DECEMBER 31, 1951

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$18,907	$20,936	$11,024	348.7	$4,998	$2,372	98.9	$18,948	$9,516	370.1	$44,882	$22,912	817.7
Arizona	6,279	2,221	1,591	43.8	532	266	.3	7,824	5,409	120.6	10,577	7,266	164.7
Arkansas	8,596	8,336	4,544	328.0	6,867	3,396	226.4	11,991	6,190	322.2	27,194	14,130	876.6
California	24,327	18,871	9,529	91.0	2,838	1,581	13.8	69,747	33,123	260.2	91,456	44,233	365.0
Colorado	9,109	3,698	2,086	88.2	1,422	805	43.6	11,376	6,056	201.7	16,496	8,947	333.5
Connecticut	7,464	3,350	1,725	11.7	1,427	710	5.1	11,705	6,099	15.3	16,482	8,534	32.1
Delaware	3,234	3,563	1,866	16.7	625	317	18.3	2,706	1,347	19.6	6,894	3,530	54.6
Florida	10,349	11,139	5,762	218.1	7,382	3,808	162.1	16,394	8,200	297.6	34,915	17,770	677.8
Georgia	13,643	12,799	6,546	319.0	11,545	5,266	117.2	27,756	13,896	426.8	52,100	25,708	863.0
Idaho	7,639	10,801	6,752	333.0	1,153	703	36.1	3,841	2,487	84.8	15,795	9,942	453.9
Illinois	37,404	28,396	15,980	207.3	17,273	8,725	141.6	69,683	35,144	587.7	115,352	59,849	936.6
Indiana	22,595	27,953	13,623	144.6	8,965	4,751	70.9	17,709	9,023	191.7	54,627	27,397	407.2
Iowa	10,622	10,411	5,613	307.8	5,895	2,966	183.8	14,114	7,024	585.9	30,420	15,603	1,077.5
Kansas	13,997	8,516	4,038	1,152.8	4,789	2,806	340.8	15,686	7,823	596.8	28,991	14,667	2,090.4
Kentucky	9,723	16,257	8,522	229.6	6,973	3,561	203.4	9,866	4,919	129.5	33,096	17,002	562.5
Louisiana	9,067	16,046	7,673	76.7	8,913	4,153	79.6	17,473	8,757	131.7	42,432	20,583	288.0
Maine	4,986	6,061	3,436	32.6	1,198	567	8.6	7,771	3,934	71.1	15,030	7,937	112.3
Maryland	9,485	1,797	815	22.1	3,351	2,148	3.4	9,241	4,221	25.0	14,389	7,184	50.5
Massachusetts	9,037	10,136	4,349	18.6	2,190	894	6.8	60,474	29,269	50.8	72,800	34,512	76.2
Michigan	17,562	20,166	10,091	385.3	8,457	4,163	141.0	48,233	20,579	247.9	76,856	34,833	774.2
Minnesota	15,287	6,481	3,436	827.8	1,245	607	133.2	21,379	11,678	690.3	29,105	15,721	1,651.3
Mississippi	12,537	8,721	4,487	330.4	3,414	1,678	80.4	17,526	9,117	447.5	29,661	15,282	858.3
Missouri	19,711	24,227	12,462	680.3	11,913	5,872	236.0	34,810	18,328	620.8	70,950	36,662	1,537.1
Montana	11,821	10,931	6,555	361.1	4,129	2,710	59.9	15,018	8,843	231.7	30,078	18,108	652.7
Nebraska	13,296	14,245	7,422	534.3	5,819	2,874	120.3	15,764	7,820	511.1	35,828	18,116	1,165.7
Nevada	7,134	3,932	2,987	113.4	745	624	14.6	3,286	2,721	133.1	7,963	6,332	261.1
New Hampshire	3,803	2,325	1,348	12.2	1,297	646	5.5	5,030	2,505	34.8	8,652	4,499	52.5
New Jersey	10,743	7,073	3,417	16.7	5,513	2,756	4.2	24,414	11,883	18.6	37,000	18,056	39.5
New Mexico	7,165	2,241	1,434	97.0	3,311	2,129	69.2	10,579	6,765	196.3	16,131	10,328	362.5
New York	52,635	73,042	38,359	152.2	20,447	9,308	122.1	89,900	41,191	297.2	183,389	88,858	571.5
North Carolina	11,410	22,226	11,028	538.9	2,569	1,264	60.4	19,867	9,989	369.7	44,662	22,281	969.0
North Dakota	6,964	6,635	3,442	1,047.0	4,299	2,139	448.3	7,703	3,887	640.7	18,637	9,468	2,136.0
Ohio	26,273	22,442	11,080	169.7	18,989	9,478	91.5	69,969	35,464	255.9	111,400	56,022	517.1
Oklahoma	10,931	10,349	5,857	187.4	6,378	3,333	74.1	16,539	8,898	163.6	33,266	18,088	425.1
Oregon	4,394	4,725	2,797	42.6	4,036	2,356	89.0	13,260	7,256	116.5	22,021	12,409	248.1
Pennsylvania	27,171	18,923	9,462	29.6	16,879	8,412	27.2	59,878	29,707	175.5	95,680	47,581	232.3
Rhode Island	4,273	4,639	2,319	36.9	906	453	7.0	14,989	7,620	12.5	20,534	10,392	56.4
South Carolina	6,575	12,136	6,634	390.6	2,041	963	115.2	11,882	5,966	174.4	26,059	13,563	680.2
South Dakota	6,743	3,750	2,170	362.7	2,723	1,652	208.8	12,321	7,113	676.8	18,794	10,935	1,248.3
Tennessee	12,184	9,365	4,359	301.8	9,265	4,305	124.9	24,832	11,837	291.4	43,462	20,501	718.1
Texas	31,101	4,884	2,319	143.4	11,865	6,031	351.2	48,677	25,396	422.7	65,426	33,746	917.3
Utah	5,710	3,645	2,763	86.2	2,979	2,175	40.7	4,158	3,052	60.1	10,782	7,990	187.0
Vermont	2,053	5,447	2,698	61.8	395	202	4.3	5,514	2,738	36.1	11,356	5,638	102.2
Virginia	12,543	17,340	8,594	381.9	5,531	2,795	117.2	20,018	9,979	332.0	42,889	21,368	831.1
Washington	9,108	8,388	3,430	129.5	2,575	1,213	13.6	17,909	9,287	101.8	28,872	13,930	244.9
West Virginia	7,989	9,806	4,947	106.0	4,302	2,162	53.4	10,143	5,057	111.7	24,251	12,166	271.1
Wisconsin	14,103	23,595	12,670	375.9	6,208	3,109	92.5	17,243	8,602	401.4	47,046	24,381	869.8
Wyoming	4,764	460	393	3.4	1,706	1,113	24.2	5,727	3,703	134.5	7,893	5,209	162.1
Hawaii	3,064	5,278	2,341	7.3	2,365	1,102	3.3	9,998	3,879	34.0	17,641	7,322	44.6
District of Columbia	6,388	2,894	1,447	.7				3,592	1,989	1.6	6,486	3,436	2.3
Puerto Rico	6,325	10,161	4,735	45.0	1,205	528	4.4	10,552	4,860	36.4	21,918	10,123	85.8
TOTAL	628,223	601,759	312,957	11,949.3	271,842	137,947	4,698.3	1,095,015	550,146	12,467.7	1,968,616	1,001,050	29,115.3

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