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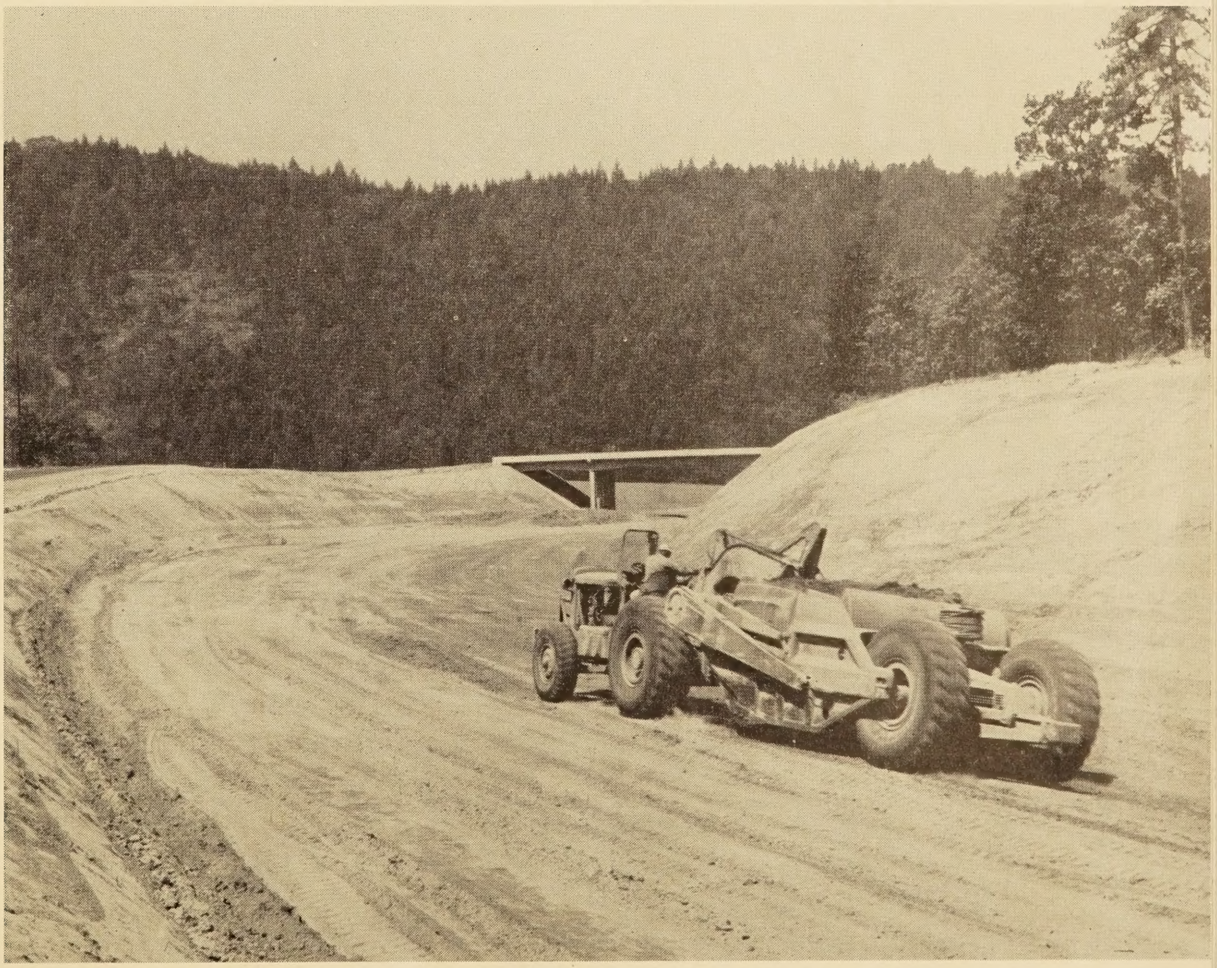

Grading of Interstate system highway, U. S. 99, between Roseburg and Myrtle Creek, Oreg.

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Pore Pressures in Base Courses

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported[1] by EDWARD S. BARBER and GEORGE P. STEFFENS, Highway Physical Research Engineers

Although it is desirable to keep roadway base courses dry, they usually get wet in spite of efforts to prevent entrance of water or to provide drainage. However, the stability of granular material depends more on the state of stress in the pore water than on the amount of water present.

A load applied to wet soil (total pressure) is carried between the soil grains (intergranular pressure) or on the water (pore pressure). Since the strength of soil depends upon the intergranular pressure, any pore pressure reduces the strength because the intergranular pressure equals the total pressure minus the pore pressure.

Therefore, measures which prevent the development of pore pressures may maintain the stability of a base course even though the material stays wet. This requires minimizing the amount of fines without making a material too harsh to place with a uniform high density.

DURING the spring and summer on sunny days, water was often observed on the surface of an experimental flexible pavement (*1, 2*)[2] built at Hybla Valley, Va. Beginning about 2 years after construction, water appeared during the early afternoon and disappeared in the evening. This water movement is not attributable to fluctuations of the water table in the subgrade because the test sections of pavement were built on a 5-foot clay fill, designed to provide uniform subgrade conditions during the period that static and moving load tests were scheduled. A similar phenomenon had been reported in Texas (*3*) and was attributed to pore pressures related to temperature changes in the base course. Related effects of pore pressures are given in published reports for bituminous surfaces (*4*), roofs (*5*), and paint (*6*).

To study this condition quantitatively, concurrent records were made of barometric pressures and temperatures and manometer pressures at various depths below the surface of the test pavement. Also fluctuations in the height of the water table in the base course were observed in holes dug through the pavement, and measurements of the height were made simultaneously with other readings for correlation purposes. Ample data on the moisture-density of the base course were available for correlation, because this study was conducted during the same period that load tests were made to evaluate the thicknesses of the various sections of flexible pavement.

This article presents the theoretical relation between temperature and pore pressure, its correlation with field measurements, and general observations of the effect of pore pressures on the stability of base courses.

Effects of Temperature Changes on Air-Water Mixtures

At temperatures above freezing, the voids in a base course are filled with air carrying various degrees of water vapor and water with dissolved air. The volume of the air-water components for any temperature and pressure condition can be calculated from the gas laws of Boyle, Charles, Dalton, and Henry. For example, assume a condition in a base course where the volume of voids for a given volume of base is 100 cc.; also assume that these voids are filled with 88 cc. of water saturated with dissolved air and 12 cc. of air saturated with water vapor at 32° F. and a pressure of one atmosphere (760 mm. of mercury).

From the properties of water taken from the International Critical Tables (*7*), shown in table 1, and molecular weights of 29 and 18 for air and water, respectively, the following calculations for several assumed temperatures and pressures indicate how the data used later in this article were developed for correlation purposes.

Using a specific volume of water 1.00013 (see table 1 for 32° F.), the weight of 88 cc. of water is 87.989 g. The vapor pressure at 32° F. is 4.58 mm., which leaves a partial pressure on the dry air of 755.42 mm. The weight of dissolved air by Henry's law, using the solubility of air in water (partial pressure per mole fraction) at 32° F. of 3.277 x 10^7, is 87.989 x

Table 1.—Physical properties of water

Temperature		Specific gravity	Specific volume	Viscosity	Surface tension	Saturated vapor		Air solubility, partial pressure per mole fraction
° F.	° C.					Pressure	Density	
				Dyne-sec./cm.² × 10^{-3}	*Dynes/cm.*	*Mm. Hg.*	*Mg./cc.*	*Mm. Hg.* × 10^7
32	0	0.99987	1.00013	17.94	75.64	4.579	0.00485	3.277
33.8	1	.99993	1.00007	17.32	75.50	4.926	.00519	3.361
35.6	2	.99997	1.00003	16.74	75.35	5.294	.00556	3.450
37.4	3	.99999	1.00001	16.19	75.21	5.685	.00595	3.536
39.2	4	1.00000	1.00000	15.68	75.07	6.101	.00636	3.624
41	5	.99999	1.00001	15.19	74.92	6.543	.00680	3.712
42.8	6	.99997	1.00003	14.73	74.78	7.013	.00726	3.803
44.6	7	.99993	1.00007	14.29	74.64	7.513	.00775	3.894
46.4	8	.99988	1.00012	13.87	74.50	8.045	.00827	3.985
48.2	9	.99981	1.00019	13.48	74.36	8.609	.00882	4.076
50	10	.99973	1.00027	13.10	74.22	9.209	.00941	4.168
51.8	11	.99963	1.00037	12.74	74.07	9.844	.01002	4.257
53.6	12	.99952	1.00048	12.39	73.93	10.518	.01067	4.347
55.4	13	.99940	1.00060	12.06	73.78	11.231	.01135	4.438
57.2	14	.99927	1.00073	11.75	73.64	11.987	.01206	4.525
59	15	.99913	1.00087	11.45	73.49	12.788	.01283	4.612
60.8	16	.99897	1.00103	11.16	73.34	13.634	.01364	4.701
62.6	17	.99880	1.00120	10.88	73.19	14.530	.01447	4.789
64.4	18	.99862	1.00138	10.60	73.05	15.477	.01536	4.874
66.2	19	.99843	1.00157	10.34	72.90	16.477	.01631	4.964
68	20	.99823	1.00177	10.09	72.75	17.535	.01730	5.044
69.8	21	.99802	1.00198	9.84	72.59	18.650	.01835	5.130
71.6	22	.99780	1.00221	9.61	72.44	19.827	.01942	5.216
73.4	23	.99757	1.00244	9.38	72.28	21.068	.02058	5.297
75.2	24	.99733	1.00268	9.16	72.13	22.377	.02178	5.379
77	25	.99707	1.00293	8.95	71.97	23.756	.02304	5.468
86	30	.99568	1.00434	8.00	71.18	31.824	.03035	5.858
95	35	.99406	1.00598	7.21	70.38	42.175	.03960	6.249
104	40	.99225	1.00782	6.54	69.56	55.324	.0511	6.611
113	45	.99024	1.00985	5.97	68.74	71.88	.0656	6.916
122	50	.98807	1.01207	5.49	67.91	92.51	.0832	7.188
131	55	.98573	1.01448	5.07	67.05	118.04	.1046	7.43
140	60	.98324	1.01705	4.70	66.18	149.38	.1305	7.64
149	65	.98059	1.01979	4.37	65.3	187.54	.1615	7.83
158	70	.97781	1.02270	4.07	64.4	233.7	.1984	7.98
167	75	.97489	1.02576	3.81	63.5	289.1	.2421	8.09
176	80	.97183	1.02899	3.57	62.6	355.1	.2938	8.17
185	85	.96865	1.03237	3.36	61.7	433.6	.3541	8.21
194	90	.96534	1.03590	3.17	60.8	525.8	.4241	8.22
203	95	.96192	1.03959	2.99	59.8	633.9	.505	8.20
212	100	.95838	1.04343	2.84	58.8	760.0	.598	8.16

[1] This article was presented at the 37th Annual Meeting of the Highway Research Board, Washington, D. C., January 1958.

[2] Italic numbers in parentheses refer to references on page 62.

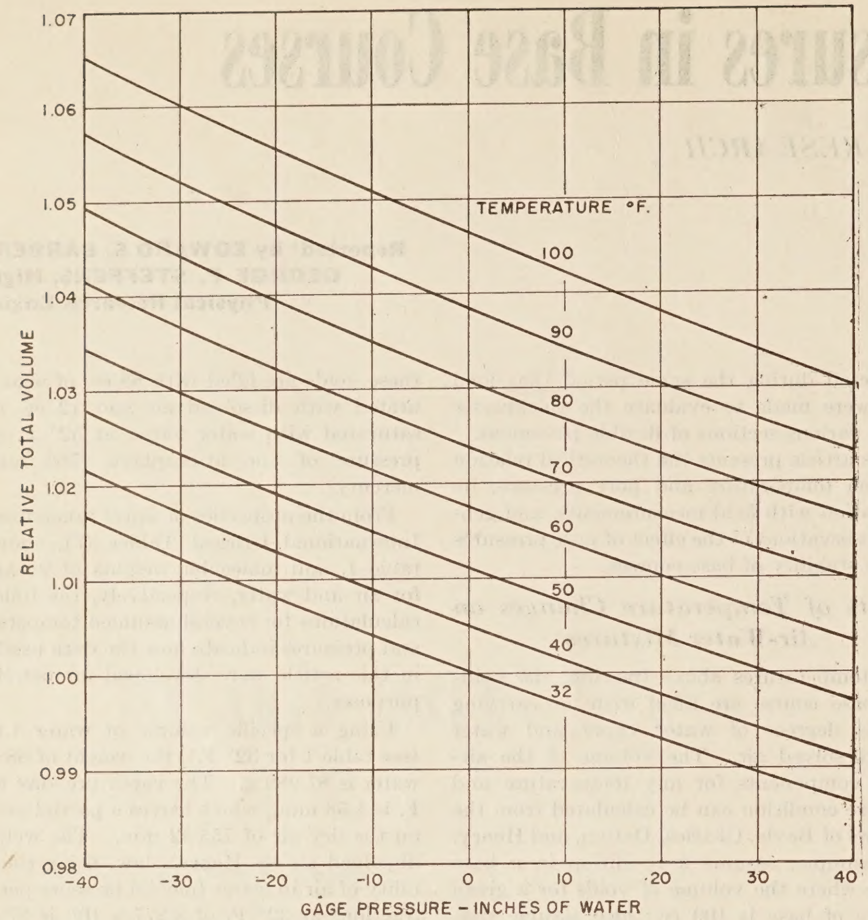


Figure 1.—Relation of volume and pressure for 0.88 volume of water and 0.12 volume of air at 32° F. and zero gage pressure.

29/18 x 755.42/(3.277 x 10^7) = 0.00327 g. Since a molecular weight of air occupies 22,400 cc. at 760 mm., the density of dry air by Boyle's law is 29/22,400 x 755.42/760 = 0.001287 g./cc. Using this value, the weight of dry air in 12 cc. of saturated air is 0.001287 x 12 = 0.01544 g.

If the temperature is raised to 68° F. and the total pressure remains constant at 760 mm., the volume of water becomes 88.145 cc., the vapor pressure becomes 17.54 mm., and the partial pressure of the dry air is reduced to 742.46 mm. With the solubility coefficient decreased to 5.044 x 10^7, the weight of the dissolved air becomes 0.00209 g. and releases 0.00118 g. to make the total free air 0.01662 g. At 68° F. the density of dry air by Charles' law is 29/22,400 x 742.46/760 x 492/528 = 0.001180 g./cc. The volume of dry air, using this calculated value of density, is 0.01662/0.001180 = 14.080 cc. Thus the total volume becomes 102.23 cc. (88.145 + 14.080), which is an expansion of 2.23 percent in the air-water mixture when the temperature is changed from 32° F. to 68° F.

Using a coefficient of cubical expansion of stone 18 x 10^{-6} per degree F., this 36° F. temperature change would cause, at the most, an expansion of 0.07 percent (18 x 10^{-6} x 36) which is negligible compared to the 2.23-percent increase of volume in the air-water mixture from 32° F. to 68° F.

If the temperature is held constant at 68° F. while the pressure is increased 25.4 mm. (13.6 inches of water), the dissolved air in water is increased by 0.00209 x 25.4/742.46 = 0.00007 g. leaving 0.01655 g. of dry air. The volume of this weight of free dry air is 0.01655/0.001221 = 13.56 cc., and therefore the total volume of water and air becomes 101.71 cc.

Similar calculations for various temperatures and pressures provide data for figures 1, 2, and 3, which show graphically the relations between temperature, pressure, and volume for an air-water mixture of constant composition. In these calculations, zero gage pressure is taken as 760 mm. of mercury.

The rates of change of volume with temperature for various air-water compositions are shown in figure 4, and figure 5 shows the amount of free air at various temperatures for several air-water compositions. The relation between pressure increase and temperature increase for various compositions at a constant volume with zero initial gage pressure is shown in figure 6. In figure 4, it is noticed that the volume increase per degree F. increase becomes greater as the relative amount of free air increases. On the other hand, figure 6 shows that the pressure increase per degree F. increase is at a maximum for zero free air and decreases with greater amounts of free air.

Pore Pressures Related to Cyclical Temperature Changes

If the temperature varies with time, and conditions are such that there is no movement of water and escape of air from a base course, the changes in pore pressure will be proportional to the temperature change shown in figure 6 for various air-water mixtures. However, if the pressure changes in the voids and the conditions are such that the downward movement of air or water from a wide thin dense base course is obstructed, either by relatively impervious fine-grained subgrade material or a frozen saturated sand, it will slowly drain laterally with the tendency to move upward through or into the upper part of a less saturated base.

This latter condition simulates a system corresponding to that occurring in a one-dimensional consolidation test with one impervious face, i. e., impervious boundaries at the bottom and sides of the base course and a reservoir of high permeability at the top (surface), except that the coefficient of consolidation, c_v, is taken as the permeability divided by the compressibility of the air-water mixture in an incompressible base course, instead of the permeability divided by the compressibility of the base course with incompressible water. Let the base (thickness of H) be subjected to a sinusoidal temperature change of unit period with uniformity in temperature changes from top to bottom of the base, such that with no drainage the pressure would be $2\pi(t+x)$ as shown in figure 7.

Under these conditions, consider an increment of applied pressure, $dp = 2\pi \cos 2\pi(t+x)dt$ which is constant to time zero; at time zero, the effect of dp is reduced by the degree of consolidation to dp'. From equation 46, page 16 of Public Roads Magazine for March 1937, the reduced pressure is:

$$dp' = 2\pi \cos 2\pi(t+x) \sum_{n=1,3,5\ldots} \frac{4}{\pi n} e^{-\frac{n^2\pi^2 c_v t'}{4H^2}} \sin\frac{n\pi z}{2H}\, dt$$

where $t'(=-t)$ is the elapsed time from pressure application to time zero and z is the depth below the upper permeable boundary. Considering the impervious boundary $(z=H)$ and expanding $\cos 2\pi(t+x)$ to $(\cos 2\pi t \cos 2\pi x - \sin 2\pi t \sin 2\pi x)$, the total resulting pressure at time zero, assuming the application of pressure to have started at minus infinity, is:

$$p' = \sum_{m=1,-3,5\ldots} \frac{8}{m} \int_{-\infty}^{0} (\cos 2\pi t \cos 2\pi x - \sin 2\pi t \sin 2\pi x) e^{\frac{m^2\pi^2 c_v t}{4H^2}}\, dt.$$

By integration

$$p' = \sum_{m\ 1,-3,5\ldots} \frac{8}{m} \frac{\frac{m^2\pi^2}{4}\frac{c_v}{H^2}\cos 2\pi x + 2\pi \sin 2\pi x}{m^4 \frac{\pi^4}{16}\left(\frac{c_v}{H^2}\right)^2 + 4\pi^2}$$

or $p' = A \cos 2\pi x + B \sin 2\pi x$

where A

$$\sum_{m=1,-3,5\ldots} \frac{\frac{m}{2}\left(\frac{c_v}{H^2}\right)}{1+\left(\frac{c_v}{H^2}\right)^2 \frac{\pi^2}{64} m^4}$$

and B

$$\sum_{m=1,-3,5\ldots} \frac{4/\pi m}{1+\left(\frac{c_v}{H^2}\right)^2 \frac{\pi^2}{64} m^4}$$

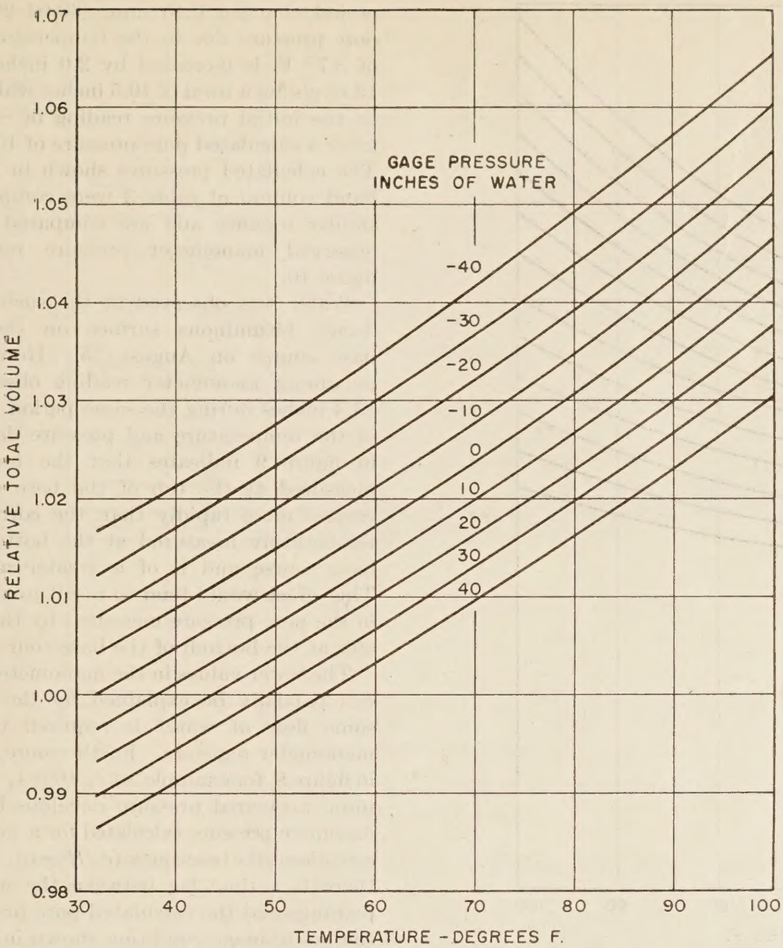


Figure 2.—Relation of volume and temperature for 0.88 volume of water and 0.12 volume of air at 32° F. and zero gage pressure.

A and B have been numerically calculated for various values of c_v/H^2 as shown in table 2. By combination, $p' = \sqrt{A^2+B^2} \sin (2\pi x + \arctan A/B)$ which is a sine wave with an amplitude of $\sqrt{A^2+B^2}$ and a maximum occurring $(\arctan A/B)/2\pi$ before the maximum for no drainage. The calculated amplitudes $(\sqrt{A^2+B^2})$ and phase displacements $(\arctan A/B)/2\pi$ of the resultant sinusoidal pressure curves for various values of c_v/H^2 are shown in table 2.

In figure 8 the sinusoidal pressure curve for no drainage ($c_v/H^2=0$) is shown starting at time zero with a pressure of zero increasing to a maximum relative pressure of 1.00 at 0.25 cycle and then decreasing to zero pressure at 0.50 cycle. This curve is shown because it is used as a base for calculating the relative amplitudes and phase displacements for other values of c_v/H^2. The peaks of the curves for various drainabilities are shown in figure 8 by the points on the line denoted as locus of maximum pressures. For comparative purposes a complete half-cycle is also shown for $c_v/H^2=1$.

For zero permeability ($c_v/H^2=0$) there is, of course, no drainage of pore water and for very high permeability (high c_v/H^2) the pressure is lost by the escape of water. It is noted that when c_v/H^2 is less than 2, the maximum pressure is greater with drainage than with no drainage. Also the maximum pressure precedes the maximum temperature (pressure with no drainage) when c_v/H^2 is greater than 0.3.

Temperature and Pore Pressure Measurements

A flexible pavement, consisting of various thicknesses of gravel base (8 percent passing No. 200 sieve after construction) and a bituminous concrete surface, was constructed over a well-compacted fill of relatively impervious clay at Hybla Valley by the Bureau of Public Roads. A comprehensive testing program of static and moving loads was conducted over a period of about 5 years. Several years after the pavement was built, a series of pressure and water table fluctuation measurements were made in the base course because of the movement of free water from the base course through the bituminous concrete surfacing during the spring and summer.

Temperatures at various depths were measured with thermistors, and water pressures were measured with water manometers. Measurements of the fluctuations in the height of the free water table in the base course were recorded simultaneously with pressure and temperature readings to assemble data from which the graphs shown in figure 9 were developed (*8*). A study of the data shown in figure 9 indicates that the temperatures measured at the top and bottom of the base course are cyclic in nature, and that the amplitude of the temperature fluctuations decreases with depth below the top of the base course. The temperature at the bottom of the base course lags behind the top of the base because of the insulating effect of the base course.

The pore pressure, measured by the manometers, and the water level observed in the

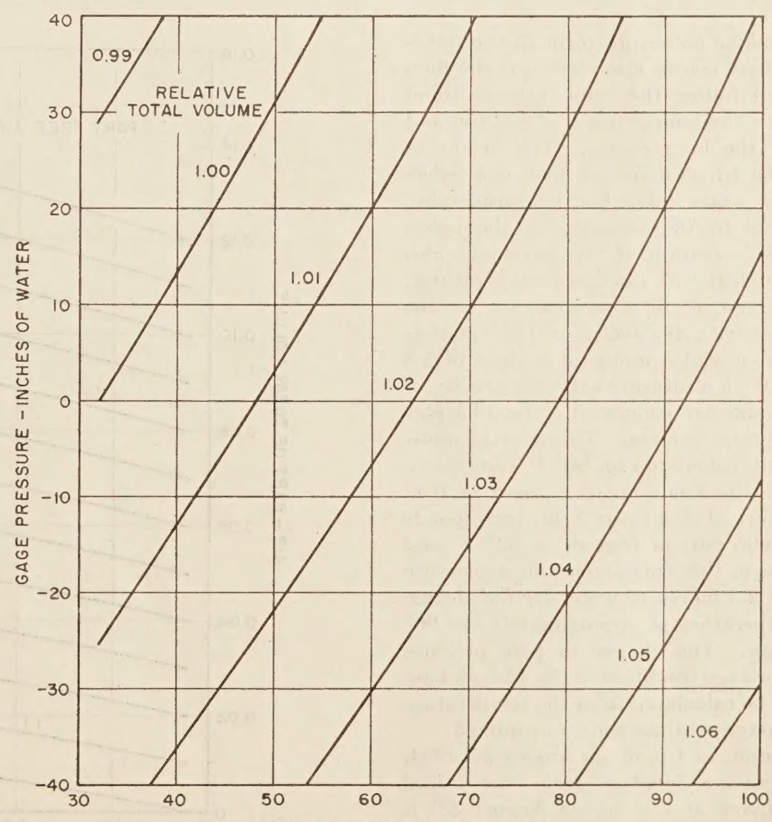


Figure 3.—Relation of pressure and temperature for 0.88 volume of water and 0.12 volume of air at 32° F. and zero gage pressure.

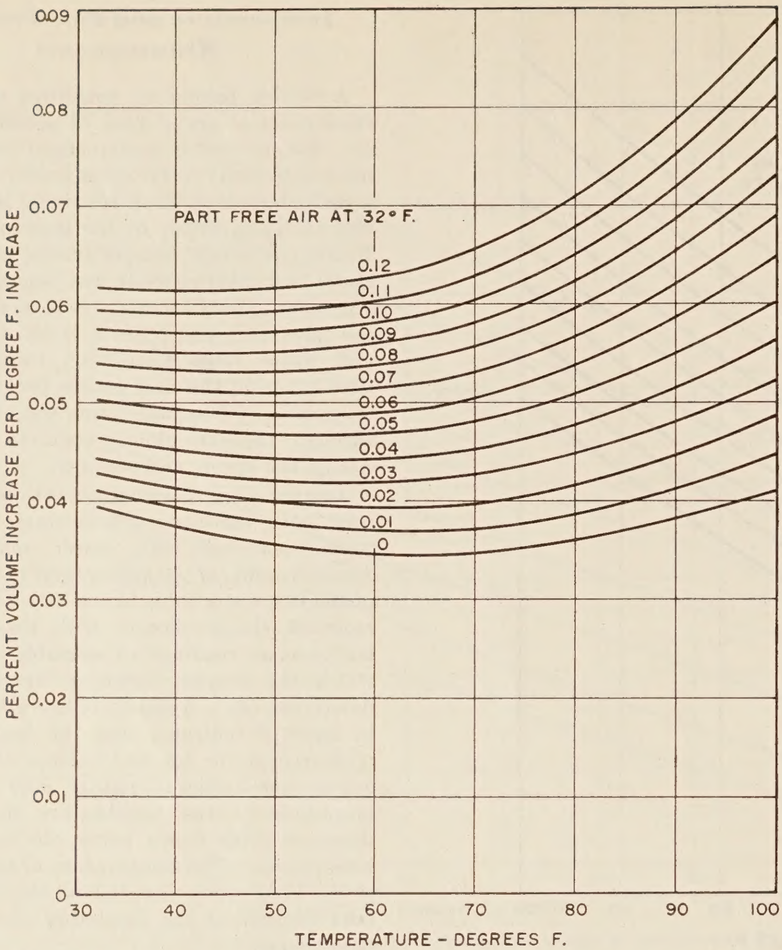


Figure 4.—Volume increase with temperature at zero gage pressure for various air-water mixtures.

4-inch diameter holes dug through the pavement and base course also show a cyclic fluctuation and follow the same general trend observed for the temperatures at the top and bottom of the base course. The height of water in the 4-inch diameter holes is a reflection of the water table but is considerably damped due to the capacity of the holes. The surface elevation of the pavement also increased slightly with increased temperatures.

The average of 16 measurements on the base course gave a dry density of 143.9 pounds per cubic foot and a moisture content of 5.3 percent. With a measured specific gravity of 2.66, the voids are computed to be 13.3 percent of the total volume. This average moisture content, calculated for 90° F. conditions, shows that the voids are comprised of 0.92 part of water. Using figure 5 this corresponds to about 0.06 part of free air at 32° F. and from figure 6, this corresponds to a pressure increase of 2.3 inches of water for one degree rise in temperature at approximately the 90° F. condition. The change in pore pressure for a no-drainage condition in the 12-inch base course can be calculated from the temperature and barometer readings shown in table 3.

For example, at 1 p. m. on August 26, 1954, the temperature increase from the initial readings (taken at 7 a. m. on August 25) is 3.7° F. (92.2–88.5); the corresponding pore pressure increase is 3.7 x 2.3=8.5. However, since the barometric pressure during this period changed 0.15 mm. (30.02–29.87), the pore pressure due to the temperature change of 3.7° F. is increased by 2.0 inches (0.15 x 13.6) giving a total of 10.5 inches which, added to the initial pressure reading of −0.3 inch, gives a calculated pore pressure of 10.2 inches The calculated pressures shown in the right-hand column of table 3 were computed in a similar manner and are compared with the observed manometer pressure readings in figure 10.

Water was observed at the surface of the 3-inch bituminous surface on the 12-inch base course on August 25. However, the maximum manometer reading observed was 13.3 inches during the same period. A study of the temperature and pressure data shown in figure 9 indicates that the temperature measured at the top of the base course increases more rapidly than the corresponding temperature measured at the bottom of the base course and is of a greater magnitude. This effect would tend to result in an increase in the pore pressure measured by the manometer at the bottom of the base course.

The lower values in the manometer readings can partially be explained by the fact that some flow of water is required before the manometer registers. Furthermore, as shown in figure 8, for example, at $c_v/H^2=1$, the maximum measured pressure develops before the maximum pressure calculated for a no-drainage condition in the base course $(c_v/H^2=0)$. Although there is a time lag between the manometer readings and the calculated pore pressures for the no-drainage condition shown in figure 10, it is obvious that there is a direct relation between temperature and pore pressure occurring in the base course. Smaller pressure

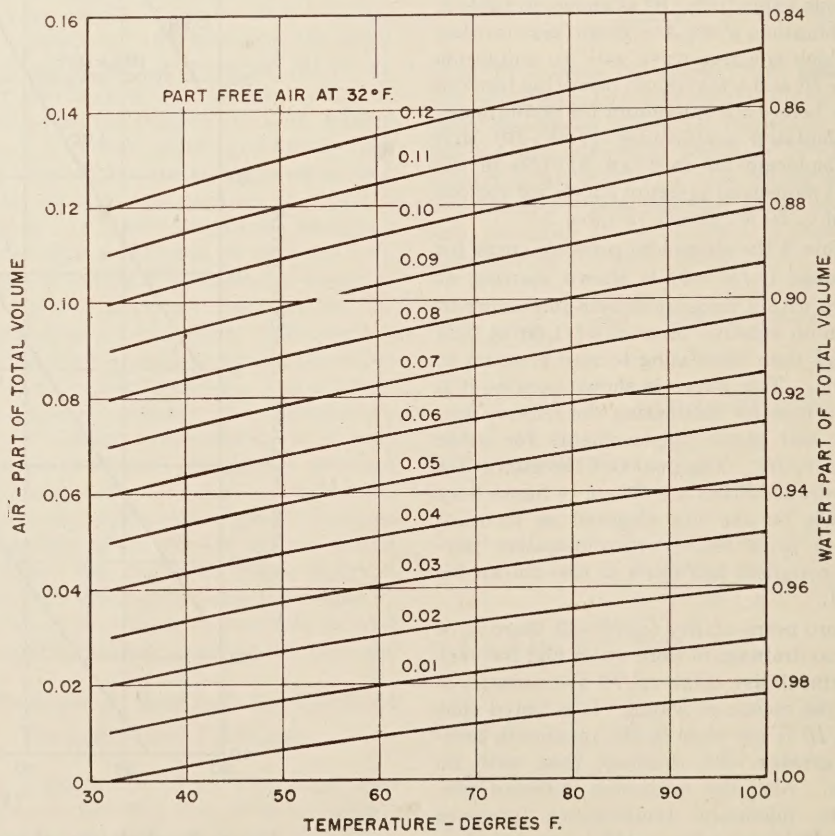


Figure 5.—Composition of air-water mixtures (zero gage pressure).

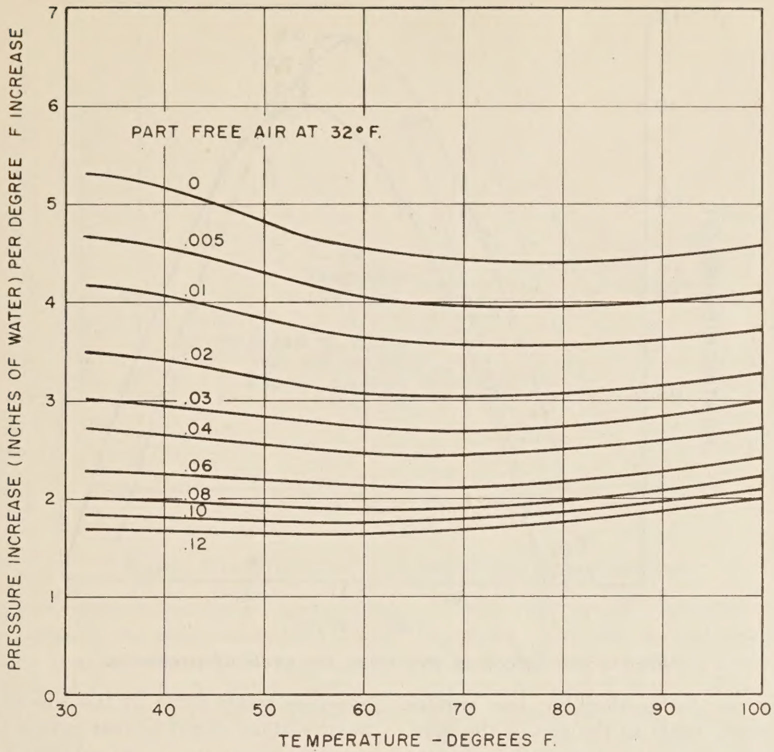


Figure 6.—Pressure increase with temperature at constant volume for various air-water mixtures.

Table 2.—Factors for cyclical pressure change

c_v/H^2	A	B	$\sqrt{A^2+B^2}$	$\frac{\text{arc tan } A/B}{2\pi}$	Abscissa of maximum
0	0	1.000	1.000	0	0.250
0.1	−0.005	.994	.994	−0.001	.251
.2	−.028	1.026	1.026	−.004	.254
.3	−.007	1.078	1.078	−.001	.251
.5	.096	1.132	1.136	.013	.237
1	.341	1.074	1.127	.049	.201
2	.569	.780	.966	.100	.150
5	.495	.261	.559	.173	.077
10	.295	.077	.304	.209	.041
20	.154	.020	.156	.229	.021
50	.063	.003	.063	.242	.008
100	.031	.001	.031	.246	.004

and temperature changes were measured in an 18-inch base course, and much higher pressure changes would be expected in a 6-inch base course because of the greater temperature changes.

A series of observations of pressure and temperature were made over a period of 4 weeks, excluding weekends, and are shown in table 4 for comparative purposes. To evaluate these data, it was necessary to make several assumptions which are as follows:

(1) Constant climatic conditions were assumed prior to August 30 and linear interpolation was used to estimate conditions over weekends.

(2) The calculations of pressure were based, as before, on a constant air-water composition.

(3) Zero time lag was assumed in making the comparison of maximum measured pressures with maximum calculated pressures.

(4) A mean pressure of one atmosphere, 760 mm. of mercury, was assumed as a base for calculating daily cyclical changes in pore pressure.

(5) To compensate for the changing daily mean temperature and barometric pressure, a 4-day lag with a parabolic decrease with time was assumed to calculate the daily effective mean temperature and barometric pressure.

This adjustment is based on the fact that after the rains from September 17 to 21, the average manometer reading was above zero for about 4 days until the excess water which entered through the surface seeped out laterally. With the assumed 4-day lag and a parabolic decrease with time, the effective temperature and barometric differential for previous days is the sum of 0.55, 0.35, 0.20, and 0.10 times the respective change in the average of the barometer and temperature changes for the previous 1, 2, 3, and 4 days.

Hourly readings were taken from 8 a. m. to 11 p. m. on each weekday for the 4-week period. The daily minimum and maximum temperatures at the bottom of the 12-inch base were obtained. Because of the limited amount of data, only about two-thirds of each daily cycle, it was necessary to use the ½ range of the daily temperature cycle (maximum—minimum)/2 and similarly the ½ range of daily barometric pressure cycle to develop data for correlation purposes.

The calculated ½ range in pressure is, as before, the sum of the barometer and temperature effect. It approximates the ½ range in manometer readings when they were determined (the manometer often failed to read the negative minimum pressure).

The temperature and barometer trends in terms of inches of water pressure are shown in table 4 and are reflected in the calculated effective temperature and barometer readings shown in figure 11. Adding the trend effects to the ½ range gives the calculated maximum pressures which are compared with the maximum manometer readings in figure 11. A good correlation is seen except for the period after rains.

The calculated values are for constant water content, so that any increase in water content due to infiltration of rainwater will temporarily make the measured pressures higher than those calculated. Thus, when the water depth in the 4-inch diameter hole rises (despite a fall in temperature) as on August 31 and September 20–22 due to the previous rain, the manometer readings are above the calculated pressures. This is especially noticeable for the last week where 3 or 4 days are required for the effects of infiltration to dissipate.

If the base course did not maintain almost a constant degree of saturation from top to bottom by capillarity, the volume change required to raise the water table would relieve much of the pressure. Figure 12 shows the moisture content after one day of drainage of a column of the passing No. 10 sieve fraction of the base course material, compacted to the equivalent density that the fraction passing the No. 10 sieve had in the compacted base course.

It is apparent from the data shown in figure 12 that the percentage of capillary water retained in the minus No. 10 sieve material in the base course is quite uniform (12.8 to 12.0 percent) for the 11-inch height above the free water table, measured for the 1-day drainage condition.

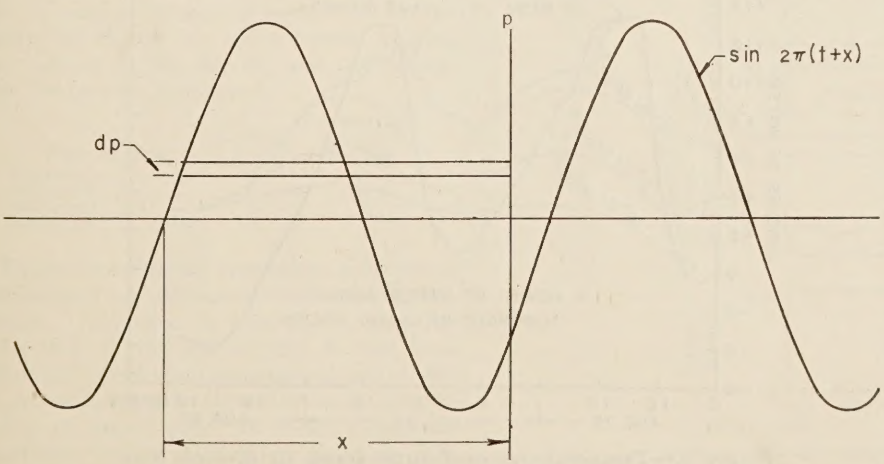


Figure 7.—Cyclical pressure applied with no drainage.

Effect of Pore Pressure on Theoretical Bearing Capacity

The effect of pore pressure on the theoretical static bearing capacity q of a base course may be calculated for various conditions from the formula and factors shown in figure 13 (*9*). A 9,000-pound wheel load on dual tires is assumed to produce a circular loaded area under the pavement surface with a radius a' of 5 inches and a pressure p' of 57 p. s. i. Assumed values for the components of the pavement are surface thickness $z=3$ inches, density of the base w' and surface $w=140$ pounds per cubic foot (0.081 lb. per cu. in.), and a coefficient of friction $f=1.0$ for a well-compacted base course.

Before applying the surface, z, a noncohesive base course material would have a bearing capacity of $q=0+0+0.081 \times 5 \times 192=78$ p. s. i. This provides very little margin of strength and makes such a material difficult to keep in place during construction. A damp stone or gravel base graded down to fine sand could develop a cohesion of 1 p. s. i. (2.3 feet of water) due to the surface tension of water. This would increase the bearing capacity to $q=1 \times 224+78=302$ p. s. i., which greatly increases its resistance to construction displacement. More cohesion could be obtained with the addition of a small amount of clay, but a slight excess would greatly reduce the friction and thereby decrease the bearing capacity when wet.

After surfacing, the moisture in the base course is apt to increase and thereby reduce the cohesion. With the water table at the bottom of a 12-inch base, the cohesion caused by surface tension would average 0.22 p. s. i. (6 inches of water). The bearing capacity would be $q=0.22 \times 224+0.081 \times 3 \times 172+78=169$ p. s. i., which is adequate to carry the imposed 9,000-pound wheel load. If a rise in temperature or leaks through the surface raised the water table to the top of the base, the cohesion would be lost; the effective density of the base w' would be reduced by the density of water and the bearing capacity would be reduced to $q=0+42+(0.081-0.036) \times 5 \times 192=85$ p. s. i., which leaves only a small margin of bearing capacity.

A temperature increase or other condition which raises the pore pressure to give an effective water table at the top of the pavement surface would further reduce the bearing capacity to $q=0+(0.081-0.036) \times 2 \times 172+(0.081-0.036) \times 5 \times 192=66$ p. s. i., which leaves very little margin of safety. Now suppose a temperature increase or an artesian condition raises the pore pressure to give an effective water table 6.7 inches above the top of the base. This reduces the surcharge effect of pavement to zero or, stated another

Table 3.—Calculation of pressure at bottom of 12-inch base course

Date, 1954	Hour, e. s. t.	Manometer reading	Temperature reading	Barometer reading	Calculated pressure[1]
		Inches of water	*° F.*	*Inches of mercury*	*Inches of water*
Aug. 25	7	−0.3	88.5	30.02	[2]−0.3
	9	−.1	87.7	30.02	−2.1
	11	2.3	87.4	30.00	−2.5
	13	8.9	88.8	29.97	1.1
	15	13.3	90.2	29.93	4.8
	17	13.1	92.0	29.89	9.5
	19	12.7	92.9	29.89	11.6
	21	9.3	93.8	29.91	13.4
	23	8.2	92.9	29.89	11.6
Aug. 26	1	6.1	92.2	29.87	10.2
	3	4.4	91.1	29.86	7.9
	5	2.9	90.8	29.87	7.0
	7	1.4	89.9	29.87	4.9
	9	.9	89.7	29.92	3.9
	11	2.4	89.4	29.92	3.2
	13	6.2	89.9	29.92	4.3
	15	12.7	91.1	29.90	7.3
	17	12.8	92.6	29.91	10.6
	19	11.5	93.5	29.92	12.6
	21	8.3	93.8	29.96	12.7
	23	5.2	93.2	29.99	10.9
Aug. 27	1	2.9	92.6	30.03	9.0
	3	1.1	91.4	30.03	6.3
	5	−.8	90.5	30.04	4.0
	7	−2.8	89.7	30.07	1.8
	9	−4.0	89.1	30.10	0
	11	−3.8	88.3	30.12	−2.2
	13	−3.4	88.5	30.13	−1.8

[1] Calculated on the basis of temperature and barometric pressure with constant air-water composition.
[2] Assumed to be equal to manometer reading.

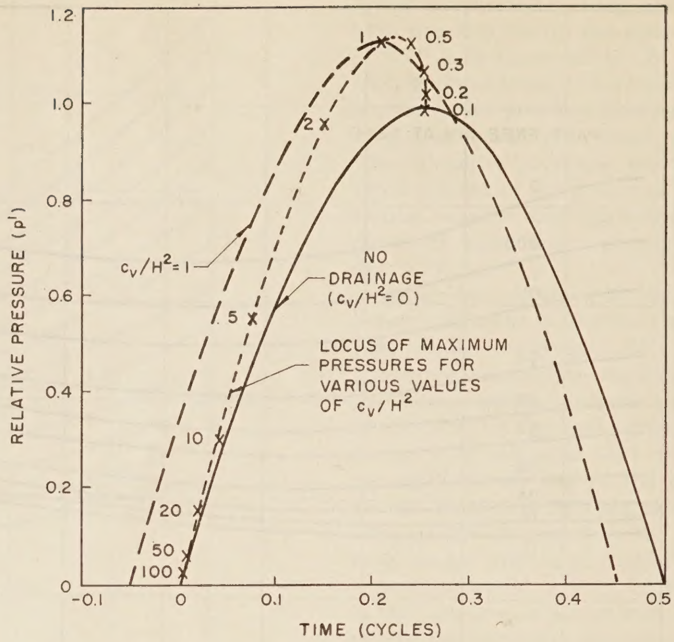


Figure 8.—Effect of drainage on cyclical pressure.

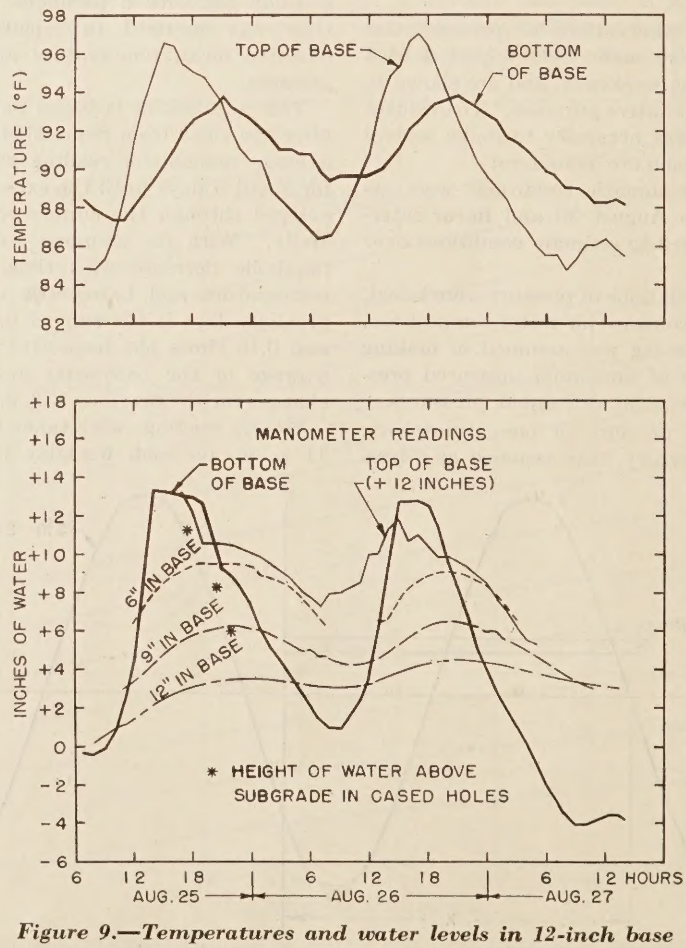


Figure 9.—Temperatures and water levels in 12-inch base course for a 55-hour period.

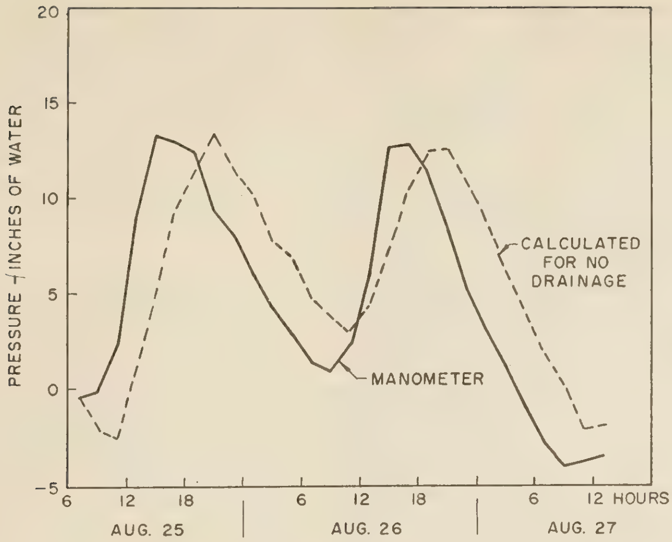


Figure 10.—Measured and calculated pore pressures.

way, the pavement is floating on a thin film of water between the base course and pavement. The bearing capacity of the base is thereby reduced to $q=0+0+(0.081-0.036) \times 5 \times 192=43$ p. s. i., which indicates inadequate bearing capacity.

These calculations for static bearing capacity of a 12-inch granular noncohesive base course (see fig. 14) show reduction of bearing capacity from 169 p. s. i. to 66 p. s. i., when the effective water table rises from the bottom of the base course to the top of a 3-inch pavement surface. This change in effective water table (pore pressure) of 15 inches could be caused by an increase of the base course temperature of 6.5° F., as shown in previous calculations.

The bearing capacities just calculated may be compared with that of an open-graded base which stays sufficiently drained so that no cohesion or pore pressure is developed. This type of base (for $f=1$ and $c=0$) would have a constant bearing capacity of $q=0(224)+(0.081)(3)(172)+(0.081)(5)(192)=120$ p. s. i.

Laboratory Tests Showing Effect of Variables

The effect of pore pressure on the strength of granular materials can be demonstrated in the laboratory. For instance, CBR tests were run on compacted graded sand with respectively 100, 62, 39, 26, and 5 percent passing the number 4, 10, 20, 40, and 200 sieves. The results were as follows:

Water condition	*Surcharge, in pounds*	*Corrected CBR at 0.1-inch penetration*
Drained	10	63
Submerged	10	36
Submerged	5	18
Submerged	0	4

Figure 15 shows the decrease in penetration resistance of wet compacted sand when rapidly heated. Less effect is observed in cohesive soils; thus, a clay soil at 120° F. had two-thirds the penetration resistance it had at 40° F., which is less than the difference in the viscosity of water (*10*). At about 85-percent saturation the air permeability increases rapidly with less saturation, as shown in figure 16.

With increasing moisture content, the optimum moisture for dynamic compaction is reached when the escape of air and water is greatly restricted during the application of load. Therefore, the locus of optimum moistures for most soils and soil-aggregate mixtures, as shown in figure 17, is at approximately 85 percent of saturation. On the other hand, an optimum moisture is not obtained for materials with very high permeability or for compaction by long-time loading.

At a density of 94 pounds per cubic foot, the subgrade under the WASHO Road Test pavement (*11*) reached a critical condition when the moisture content rose to 23 percent. With a measured specific gravity of 2.55, this is a relative saturation of 85 percent.

Samples of this soil were tested in the soils laboratory of the Bureau of Public Roads. Figure 18 shows the relation between moisture and density for the WASHO subgrade soil and the penetration (0.001 inch of a 2-inch diameter piston loaded with 83 pounds and with 5,000-load applications) obtained for various percentages of saturation. A study of these data indicates that samples compacted to less than 85-percent saturation had lower values of penetration than those compacted to more than 85-percent saturation at equivalent moisture contents. Since penetration is a measure of strength, it is obvious that the bearing capacity of this soil is critical at or near 85 percent of saturation for dynamically compacted soils. However, it should be noted that soils statically compacted at the same moisture and density (see fig. 18) have much less penetration, and therefore have greater bearing capacity. The difference in these two methods of compaction is largely due to shearing during the dynamic compaction of this soil.

In addition to the adverse pore pressure developed with the increased percentage of saturation during loading, other related factors must be considered: (1) the effect of increased

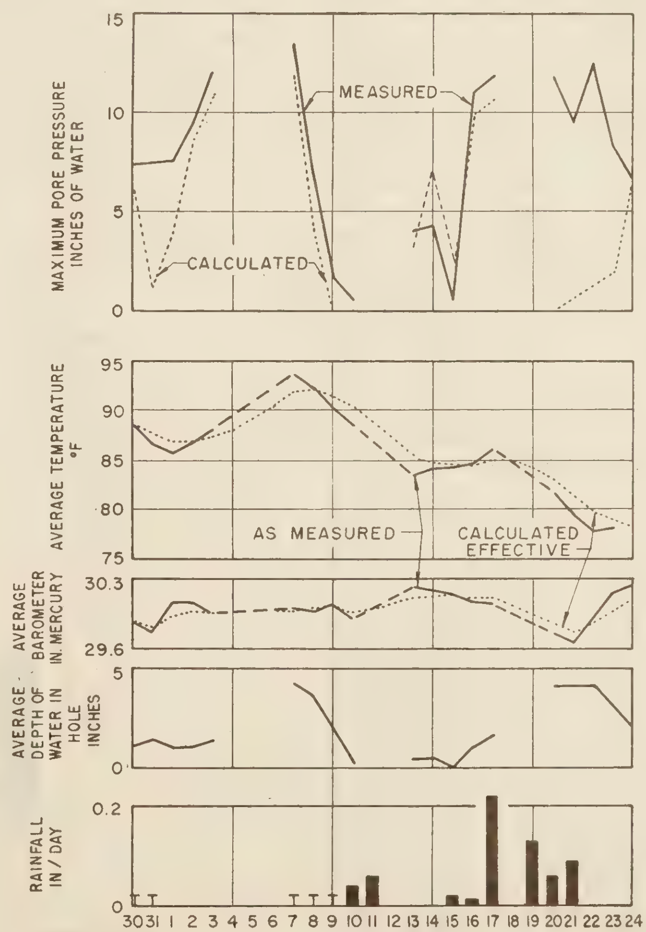


Figure 11.—Correlation of calculated pore pressure with measurements.

Table 4.—Pressures and temperatures for 4 weeks at bottom of 12-inch base course

Date	Barometer reading		Temperature reading		Calculated pressure, ½ range	Manometer reading		Temperature trend	Barometer trend	Calculated maximum pressure
	Mean, 8 to 18 hours	Difference at 18 hours	Mean, maximum and minimum	½ range x 2.3		½ range	Maximum			
	Inches of mercury	*Inches of water*	°*F.*	*Inches of water*	*Inches of water*	*Inches of water*	*Inches of water*	*Inches of water*	*Inches of water*	*Inches of water*
Aug. 1954:										
30	29.86	1.1	88.6	5.1	6.2	(1)	7.4	0	0	6.2
31	29.79	−1.0	86.7	4.3	3.3	5.2	7.5	−2.5	0.5	1.3
Sept. 1954:										
1	30.07	.4	85.6	8.6	9.0	(1)	7.6	−2.8	−1.8	4.4
2	30.08	1.0	86.7	9.4	10.4	(1)	9.5	−.4	−1.2	8.8
3	29.94	.5	88.0	8.5	9.0	(1)	12.1	1.6	.3	10.9
7	29.98	.7	93.5	7.5	8.2	8.6	13.5	3.8	−.2	11.8
8	30.01	.1	92.3	4.1	4.2	5.7	6.9	.6	−.3	4.5
9	30.06	.4	90.3	2.6	3.0	4.0	1.8	−2.6	−.6	−.2
10	29.90	1.1	88.4	2.3	3.4	(1)	.6	−4.2	.9	.1
13	30.24	1.2	83.3	8.4	9.6	(1)	4.1	−4.8	−1.5	3.3
14	30.14	.5	84.1	8.3	8.8	(1)	4.3	−1.5	−.3	7.0
15	30.17	.5	84.2	2.5	3.0	(1)	.7	−.4	−.2	2.4
16	30.08	.8	84.5	8.1	8.9	6.3	11.1	.5	.6	10.0
17	30.06	.7	86.0	7.1	7.8	8.3	11.9	2.4	.6	10.8
20	29.73	−.1	81.7	1.6	1.5	5.2	11.8	−3.3	1.7	−.1
21	29.63	1.4	79.1	2.8	4.2	5.0	9.6	−5.4	1.7	.5
22	29.93	−.3	77.8	7.4	7.1	7.7	12.5	−4.7	−1.3	1.1
23	30.19	.3	77.9	7.1	7.4	7.3	8.3	−2.4	−2.9	2.1
24	30.26	1.1	77.7	9.8	10.9	(1)	6.5	−1.4	−2.4	7.1

[1] Manometer failed to read minimum pressure.

percentages of granular material, (2) the increased coefficient of friction resulting from increase in angularity of the granular components of the soil, (3) the effect of density, and (4) the effect of varying the amounts of soil fines (passing No. 200 sieve) in granular materials.

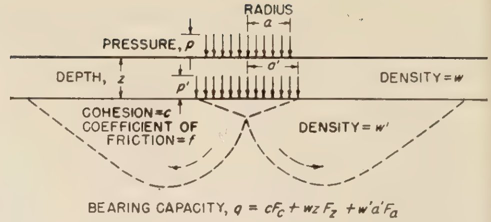
Triaxial strength test data reported for pervious granular materials (*12*) show that the total strength of these materials increases up to 20 percent when the percentage of coarse aggregate above the No. 4 sieve is increased from zero to 50 percent. Also when the angularity of the coarse material was changed from rounded to crushed particles, there was an additional 10-percent increase in the coefficient of friction. However, the largest variable affecting the strength obtained was the density at which the specimens were tested. The average coefficients of friction obtained by the triaxial compression test for materials of various gradations were 0.76, 0.85, and 0.98 for relative densities of 0.50, 0.70, and 0.90, respectively.

Another investigator (*13*) reported similar advantage of densification of well-graded materials containing 10 percent passing the No. 200 sieve. With 6-percent water and lateral pressures up to 30 p. s. i., the triaxial tests indicated relative strengths of 0.58, 0.85, and 1.00 for dry densities of 126, 136, and 141 pounds per cubic foot, respectively.

The following data taken from an unpublished portion of a report (*14*) show the effect of decreasing the percentage of material passing the No. 200 sieve on compressive strength (lateral pressure equals 20 p. s. i.) and stress-strain modulus, as measured by the triaxial compressive strength test:

Percentage passing No. 200 sieve	*Compressive strength, p. s. i.*	*Stress-strain modulus, kips/sq. in.*
15	190	5
8	210	8
0	210	11

A study of these data indicates that there is a considerable increase in the stress-strain



f	F_c	F_z	F_a
0	7.4	0.0	0.0
0.1	9	0.8	0.1
0.2	14	2.1	0.4
0.3	19	4.3	1.2
0.4	27	8	30
0.5	36	14	6.6
0.6	53	24	13
0.7	75	40	26
0.8	106	67	51
0.9	156	108	102
1.0	224	172	192

Figure 13.—Bearing capacity under a circular loaded area.

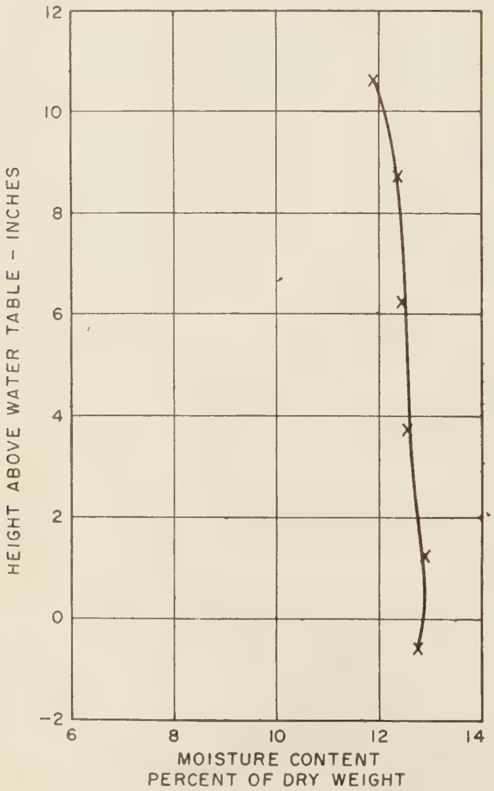


Figure 12.—Capillary water retention 24 hours after compaction.

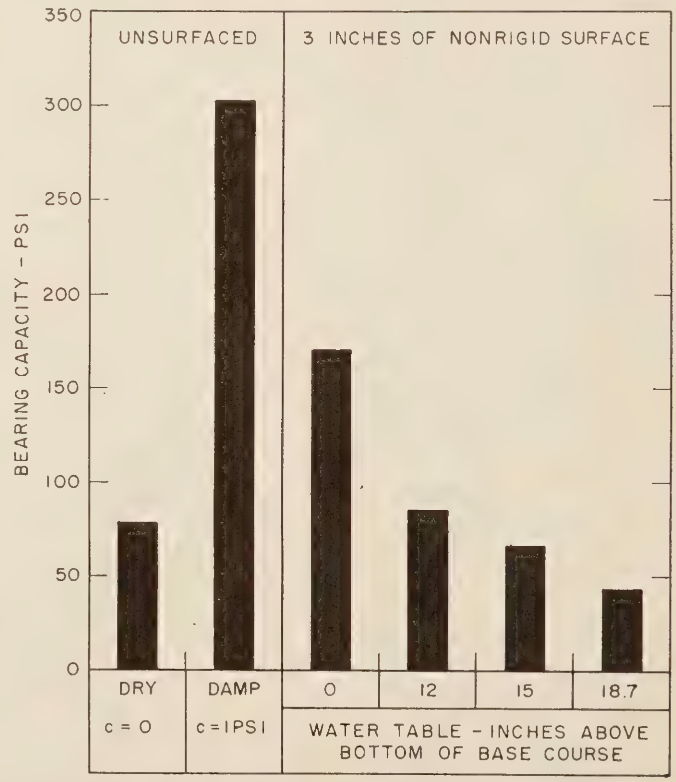


Figure 14.—Effect of pore pressure on calculated bearing capacity of a 12-inch base course (w=w'=140 lb./cu. ft., f=1.0, a'=5 inches).

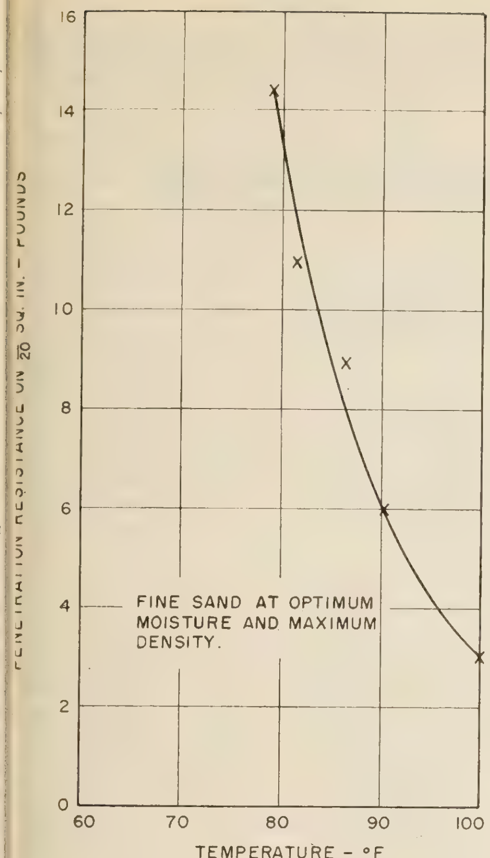


Figure 15.—Temperature related to penetration resistance.

modulus with a decrease in the percentage of soil passing the No. 200 sieve.

A third investigator (*15*) reported that a maximum density of a soil-gravel mixture was obtained with 10 percent passing the No. 200 sieve and a maximum CBR when 7 percent was used.

Field Conditions Where Pore Pressure Is a Factor

Critical pore pressures which affect the bearing capacity of a subgrade or base course can be controlled by changing the gradation and drainage so that the material, when compacted, has void spaces which are less than 80-percent saturated.

During construction, developed pore pressures may cause shearing which reduces the strength of soils in spite of the precautions taken during normal compaction. This effect may be minimized by compacting the soil at moisture contents slightly below standard AASHO optimum for low volume change soils, and by using larger roller contact areas to lower unit pressures. For earth dams, compaction 2 percent less than optimum is specified to minimize critical pore pressures in the structure. However, too low an initial moisture content in clay soils will usually cause swelling when the water content increases.

A well-drained condition requires a means of egress of water greater than the potential infiltration from the edges and through the surface. Excess water in a base may be removed downward by flow into a permeable subgrade with a deep water table or by lateral flow through a thick permeable base with sufficient outlets to reduce the required distance of lateral flow. Dense-graded bases of usual dimensions generally have insufficient permeability for rapid lateral drainage if saturated.

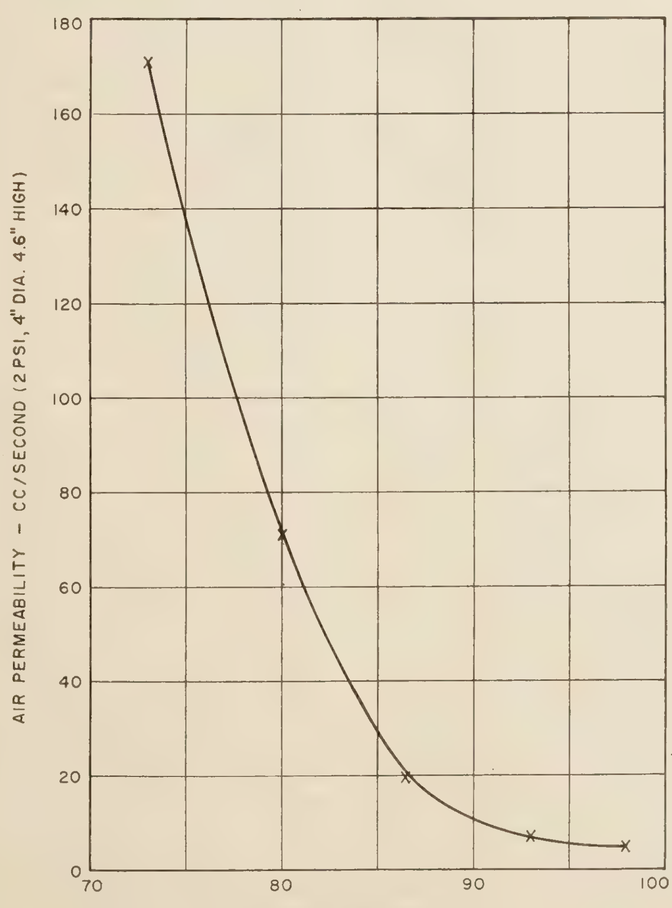


Figure 16.—Air permeability related to saturation.

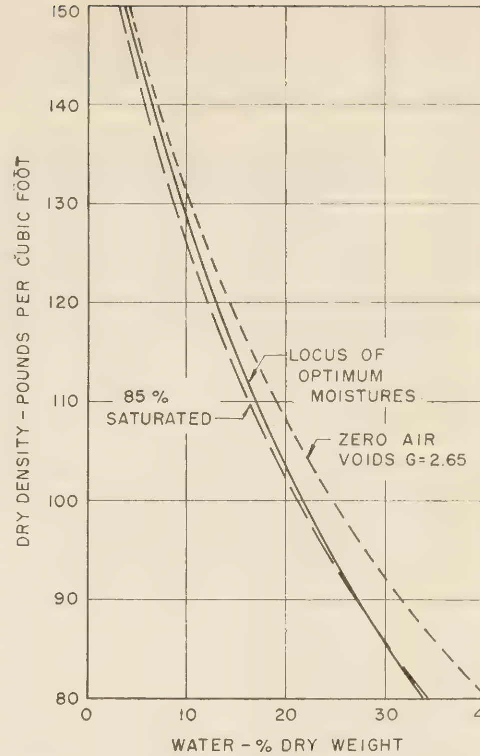


Figure 17.—Optimum moisture related to saturation.

In nonfrost areas, dense-graded materials have *sometimes* stood up even with poor drainage. This may be attributable to the tendency for a dense granular material to expand during shearing to overcome the interlocking obtained with compaction beyond the critical density. Thus, a 10° F. rise in temperature will cause a pressure of 23 inches of water, assuming 6 percent air at 32° F., but if the material expands 0.6 percent of its volume, the pressure is relieved.

This volume change is within the range of expansion of graded granular material compacted to 100 percent of standard compaction. Under dynamic loads, the effect of a standing water table may possibly be counteracted by this same expansion, if loads are not repeated at the same place, since high density with limited permeability causes a temporary vacuum to be created by shear expansion.

In regions where deep frost penetration occurs, thawing from the surface will usually prevent downward drainage of the base course for a period of time. Freezing and thawing of a very wet base course may adversely affect its stability, unless it has sufficient permeability when it thaws to allow water to escape from under a wheel load without lateral flow of the granular base. This requirement is demonstrated by materials which do not shove in the standard impact compaction test at moisture contents greater than the optimum moisture content (*16*). This criterion gener-

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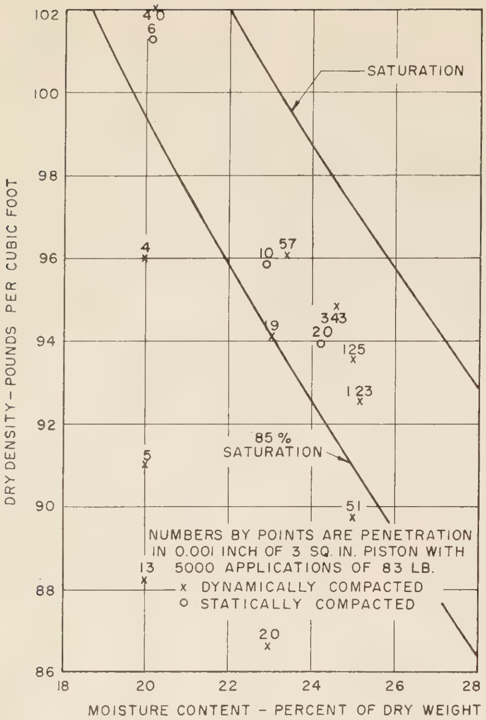

Figure 18.—Penetration related to moisture and density.

ally limits the amount of fines to a maximum of 10 percent passing the No. 200 sieve, and this limit is used in several States. In North Carolina, extra-thick surfaces are required for bases with more than 10 percent passing the No. 200 sieve (*17*). Degradation during and after construction must be considered.

If open-graded base courses are used as insulation against frost, care must be taken to prevent ventilation through the base which can nullify the insulating effect. Such ventilation is the apparent reason for increased frost penetration into the subgrade with increased pavement thickness at the WASHO Road Test Project (*11*). The density of 132 pounds per cubic foot with an average moisture content of 5 percent corresponds to a degree of saturation of 50 percent which indicates a high air permeability.

Further research is required to develop more refined design criteria for base courses. This should include the thickness, width, and slope of base courses, the permeability, capillarity, strength and stress-strain relations of base course materials, as well as the ambient conditions of moisture and temperature.

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A Rapid Method Utilizing Surface Area Measurements in Predicting the Amount of Cement Needed to Stabilize Plastic Soils

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by SIDNEY DIAMOND and EARL B. KINTER,
Highway Physical Research Engineers

The relation between the cement requirement of soil-cement mixtures and surface area determined by the glycerol retention method was investigated for a group of plastic soils. For soils containing less than 45 percent silt, a strong correlation was found between surface area and the cement content at which a 10-percent loss occurred during 12 cycles of the freeze-thaw durability test.

A regression equation and procedure were developed for predicting the cement requirement from surface area. In a comparison of predicted cement requirements with those derived from freeze-thaw test data, an average deviation of 0.6 percent cement was obtained.

THE USE of soil cement for highway base courses has increased rapidly in recent years, especially in areas where local sources of granular deposits are nonexistent or rapidly being depleted. Present methods for determining the amount of cement necessary to produce a stable material from a given soil involve the use of rather large soil samples and an extended testing program. The freeze-thaw or wet-dry durability tests (*1*)[2] in particular require considerable time and effort. A short-cut method developed by the Portland Cement Association (P. C. A.) provides an easier alternate procedure for sandy soils, but extended testing is still required for plastic materials. The present article is concerned primarily with these plastic materials, and with the development of a shorter, more economical test procedure for estimating their cement requirements.

Despite a number of years of successful use of soil cement, little is known of the physical chemistry of the reactions between the soil and the cement. It has been noted that the more plastic soils generally require larger quantities of cement for effective stabilization. Although there does not seem to be any consistent relation between cement requirement and plasticity index or, indeed, any of the familiar engineering tests, it was thought that perhaps such a relation might exist with the surface area of the soil.

Some years ago, Catton (*2*), exploring this proposed relation by the use of surface area values computed from grain-size distribution curves, found an almost complete lack of correlation. However, an examination of Catton's data reveals that the largest values he computed for surface area were just over 2 square meters per gram (m.²/g.), even for A–6 and A–7 soils. Present knowledge indicates that actual surface areas are many times higher than this, often as high as several hundred m.²/g. Catton's computed values of surface area could not therefore be expected to provide any real correlation.

The method widely considered as standard for realistically determining the surface area of fine-grained materials is known as the Brunauer-Emmett-Teller (B. E. T.) Method, and involves the adsorption of nitrogen or some other gas on the surface of the material at low temperatures. The complicated apparatus and time-consuming procedures required, however, are not suitable for general routine determinations. Further, the method is not particularly applicable to soils since the nonpolar gases normally used do not measure the internal surfaces of expanding clay minerals which are present in many soils. The crystals of the clay minerals are formed in sheetlike layers; in an expanding mineral the flat surfaces of the inner layers constitute the internal surface.

Lately, a method (*3*) involving the retention of ethylene glycol by soils has also been used to estimate surface area, but there is some doubt as to the specific quantitative relation between the actual surface area and the amount of ethylene glycol retained under the conditions of the determination. More recently, a new and simpler method (*4, 5*) involving the retention of glycerol has been developed at the laboratories of the Bureau of Public Roads.

Values of surface area obtained for various clays by this method agree closely with theoretical values of expanding clays and, for nonexpanding clays, with those determined by the B. E. T. Method. In the present work, surface areas were measured by this method on a number of soils of known cement requirement. The correlation between the resulting surface area values and cement requirements was then determined. An equation was derived for predicting cement requirements from surface area values, and the predicted cement requirements were compared with the actual cement requirements as determined by test.

Conclusions

The following conclusions are drawn from this investigation:

1. For soils of measurable plasticity, a definite correlation exists between surface area, as measured by the glycerol retention procedure, and cement requirement (percent by volume) calculated from loss data of the freeze-thaw durability test and amended by other engineering considerations. However, this correlation is not sufficiently close to permit adequate predictions to be made directly from the surface area values. The weakness of this correlation is considered to be due to such factors as:

(*a*) The surface area expressed on a weight basis in contrast to the volume basis of the cement requirement.

(*b*) The different standards of allowable losses in the freeze-thaw test, 7, 10, or 14 percent, depending on the AASHO classification of the sample.

(*c*) The use of a certain amount of engineering judgment in deriving practical recommendations for field use from the freeze-thaw data.

(*d*) The rounding of the cement requirement to the nearest one-half percent.

(*e*) The presence of several soils high in silt content in the group of samples studied. Such soils have been shown to have cement requirements which do not correlate with surface area.

2. A very strong correlation is obtained when soils of higher than 45 percent silt are

[1] This article was presented at the 37th Annual Meeting of the Highway Research Board, Washington, D. C., January 1958.

[2] Italic numbers in parentheses refer to the list of references on pages 69–70.

excluded from consideration, and the cement factor in the correlation is taken as the actual cement content by weight at which a 10-percent loss occurs in the freeze-thaw test, no allowance being made for AASHO class differences. The regression equation $y=0.087$ (surface area) $+3.79$ can be employed to derive accurate predictions of this cement factor from measurements of surface area by the glycerol retention method.

3. A suitable prediction of the conventional cement requirement, in percent by volume, can then be made by the following procedure: (*a*) modifying, if necessary, the cement factor predicted by the regression equation by adding 2.0 percent cement to adjust to the basis of a 7-percent allowable loss in the freeze-thaw test, or by subtracting 0.7 percent to adjust to the basis of an allowable loss of 14 percent; and (*b*) converting the modified value to a percent by volume basis using the density of the soil-cement mixture.

In a comparison of cement requirement values obtained by test with values obtained by the use of this procedure, the average deviation for the group of samples used in this study was 0.6 percent cement by volume, and considerably less than this for the more highly plastic soils of the group.

4. Admittedly, these results were obtained from a restricted number of samples and the procedure should be checked further with a wider and more representative selection of soils. Furthermore, use of the surface area determination to predict cement requirements should be accompanied by compressive strength or other tests on small specimens made at and near the predicted cement requirement.

Materials and Methods

Soils

Of the soils furnished for study,[3] only those with measurable plasticity indexes were included in this analysis. The soils of the first group received (group A) are described in table 1. They were not selected as representative of any particular soil area or type of soil, but were simply those on hand at the P. C. A. laboratory at the time this study was initiated. They do, however, include samples from a number of different States and of various soil horizons. The last 10 soils listed in table 1 (group B) were received as a second group and represent samples processed by the P. C. A. laboratory during the period in which this study was being made.

[3] Soil samples and accompanying test data were supplied by the Portland Cement Association. Messrs. J. A. Leadabrand and L. T. Norling of the Soil Cement Bureau were particularly helpful to the authors in conducting the present study.

Surface area measurements

In the glycerol retention method, advantage is taken of the ability of clay and other soil constituents to adsorb glycerol molecules on their surfaces. Conditions are maintained under which only a single layer of the glycerol molecules is adsorbed and retained. The amount of glycerol adsorbed is measured by weighing the sample before and after treatment, and the weight of the adsorbed glycerol can be related to the surface area of the sample.

As applied to this study, the method involves the following steps:

(1) Dry duplicate small samples (about 1 gram each) of the passing 40-mesh fraction of the soil at 110° C. in aluminum foil dishes, and

Table 1.—Description of soils

Identification						Physical properties [1]													
						Gradation: percentage finer than—													
B.P.R. soil No.	P.C.A. soil No.	State	County	Soil series	Soil horizon	4-mesh sieve	10-mesh sieve	40-mesh sieve	200-mesh sieve	0.05 mm.	0.005 mm.	0.002 mm.	Liquid limit	Plasticity index	AASHO classification	Optimum moisture of soil-cement mixture	Maximum density of soil-cement mixture	Cement requirement, P.C.A.	Surface area of whole soil
Group A Soils																			
																Percent	*Lb./cu. ft.*	*Percent by volume*	*m.²/g.*
S-32044	7538	Alabama	Madison		-------	100	85	61	24	21	6	5	17	4	A-2-4 (0)	11.4	115.5	7.0	12.5
S-32045	7494	do	Montgomery		B	100	100	98	70	48	29	28	35	11	A-6 (7)	17.7	104.0	13.5	90
S-32046	7537	do	do		B	100	100	94	43	38	29	27	34	12	A-6 (2)	17.9	108.0	10.5	49.5
S-32047	7509	Arkansas	Ashley	Richland	A, B, C	100	100	98	96	92	27	20	32	11	A-6 (9)	16.2	105.5	9.5	63
S-32050	7489	do	do	Portland	A	100	99	95	92	90	23	15	27	6	A-4 (8)	17.3	103.5	17±	59
S-32051	7490	do	do	do	B	100	100	97	94	90	23	18	30	9	A-4 (8)	18.2	104.7	11.0	79
S-32053	7552	do	Nevada	Ruston	C	100	100	70	30	28	22	18	39	17	A-2-6 (1)	15.0	109.0	7.5	48
S-32055	7555	do	do	do	A, B, C	100	99	99	38	32	22	18	20	6	A-2-4 (0)	12.7	119.0	7.0	37.5
S-32056	7542	do	White		A	100	99	97	76	56	19	12	22	5	A-4 (8)	12.3	112.6	11.0	36
S-32060	7375	Idaho	Idaho		-------	55	27	15	9	9	2	2	26	5	A-1-a (0)	10.0	134.7	9.0	14
S-32061	7497	Illinois	Cook		-------	55	47	35	19	15	7	5	25	6	A-1-b (0)	8.0	125.3	7.0	12
S-32062	7498	do	do		-------	100	99	97	91	88	42	28	41	16	A-7-6 (11)	23.4	96.7	14.5	81
S-32063	7-h	do	do		-------	100	100	97	90	87	48	--	41	24	A-7-6 (14)	17.5	104.5	13.0	77.5
S-32064	7460	do	Henry		-------	59	46	28	16	16	8	6	26	6	A-1-b (0)	9.4	130.3	7.0	7.5
S-32065	7558	do	Iroquois	Hagener	-------	85	83	78	33	22	9	3	18	7	A-2-4 (0)	10.2	121.0	7.5	25.5
S-32066	7528	do	Massac		-------	80	74	67	56	49	10	7	22	1	A-4 (4)	11.3	117.6	8.5	26
S-32067	7529	do	do		-------	77	71	66	60	58	16	13	26	6	A-4 (5)	13.3	115.8	10.5	40.5
S-32068	7530	do	do		-------	75	63	50	35	32	9	8	19	4	A-2-4 (0)	11.0	120.8	9.0	28.5
S-32069	7560	do	do		-------	55	48	41	36	34	11	8	30	9	A-4 (0)	11.2	119.5	7.0	22.5
S-32070	7561	do	do		-------	84	76	73	39	23	8	8	20	3	A-4 (1)	11.7	118.6	7.0	19.5
S-32071	7562	do	do		-------	80	71	66	56	51	15	12	26	5	A-4 (4)	13.0	117.4	9.5	37.5
S-32072	7563	do	do		-------	55	47	34	12	11	8	8	28	15	A-2-6 (0)	9.4	126.8	7.0	14
S-32074	S-7	do	McHenry		B, C	100	99	99	92	79	13	--	26	7	A-4 (8)	15.0	113.0	9.0	42.5
S-32075	6900	Kansas	Grant		-------	100	97	94	77	69	25	19	29	9	A-4 (8)	16.5	106.7	10.5	87.5
S-32076	7520	Kentucky	Carter		-------	59	56	52	24	13	4	3	19	3	A-2-4 (0)	7.8	130.2	6.5	6
S-32078	7515	Indiana	Vanderburgh		-------	100	100	94	27	26	15	12	20	4	A-2-4 (0)	12.2	120.0	7.5	20
S-32082	7525	Missouri	Jackson		C	100	100	100	94	79	17	14	27	5	A-4 (8)	15.5	108.7	9.0	82
S-32083	7526	do	do		-------	100	100	100	94	80	15	13	28	6	A-4 (8)	15.7	108.0	9.5	85.5
S-32085	7514	Oklahoma	Okmulgee		A	92	86	85	36	25	15	10	27	7	A-4 (0)	12.0	117.6	7.0	32.5
Group B Soils																			
S-32572	7682	Colorado	Fremont		-------	67	52	39	29	27	16	10	26	9	A-2-4 (0)	11.8	124.2	11.0	14.5
S-32573	7687	Illinois	Will		B	100	100	99	97	93	45	37	47	25	A-7-6 (15)	22.0	100.3	15.0	143
S-32575	7695	Louisiana	Livingston		-------	100	99	98	94	86	17	12	26	4	A-4 (8)	17.7	104.4	15.5	33
S-32576	7701	Tennessee	Franklin		B	100	99	88	53	48	32	26	32	15	A-6 (6)	16.9	109.4	10.0	36
S-32577	7702	do	do		A	98	96	83	40	36	21	17	25	10	A-4 (1)	13.7	113.5	7.0	34
S-32578	7761	Montana	Silver Bow		-------	96	83	46	22	20	10	9	30	9	A-2-4 (0)	11.8	119.5	7.5	34
S-32579	7762	Michigan	Calhoun		A, B	92	87	75	37	31	13	9	23	8	A-4 (0)	7.9	115.9	10.0	37
S-32582	7776	Kansas	Douglas		-------	100	100	100	97	85	16	14	29	5	A-4 (8)	21.0	97.0	12.0	78
S-32583	7777	do	do		-------	100	100	100	98	86	16	14	29	6	A-4 (8)	18.5	102.2	12.0	73.5
S-32584	7779	Texas	Wichita		-------	55	39	25	16	13	5	5	25	9	A-2-4 (0)	8.4	129.0	7.0	18

[1] Data furnished by the Portland Cement Association.

weigh to 0.0002 gram on an analytical balance.

(2) Add 10 ml. of a dilute (2 percent) water solution of glycerol to the sample, and swirl the container gently to mix the contents.

(3) Heat at 110° C. (±3°) in a mechanical-convection oven containing a supply of glycerol to provide a source of free glycerol vapor in the oven chamber. Under these conditions, glycerol in excess of a monomolecular layer and water are both removed.

(4) Reweigh after equilibrium has been attained, normally after overnight heating. The gain in weight over the original oven-dry weight of the sample is due to the monomolecular layer of glycerol adsorbed on both the internal and external surfaces. The adsorbed glycerol is expressed as a percentage of the 110° C. dry weight of the soil.

(5) A distinction must be made between that portion of the glycerol retained on external surfaces of all clay minerals and that retained on internal surfaces of expanding minerals such as montmorillonite and vermiculite. On the internal surfaces both the top and bottom of the monomolecular layer of glycerol are in contact with clay surfaces. On the outside of the particles, however, only one side of the monomolecular layer is in contact with the clay surface. Therefore, a given amount of glycerol on internal surfaces accounts for the twice as much clay surface area as the same amount would if it were on external surfaces.

To make this distinction, a second determination is required. This is accomplished by determining the percentage of glycerol retained by replicate samples previously heated to 600° C., the glycerol retention being determined by the same procedure previously described. Heating to 600° C. normally collapses and irreversibly closes the internal spaces and thus renders them inaccessible to glycerol molecules. The difference between the original percentage of glycerol retained and that retained after heating to 600° C. is attributable to internal surfaces; the percentage measured after this preliminary heating is due to external surfaces only.

Based on X-ray diffraction evidence concerning the thickness of a monomolecular layer of glycerol, it has been shown (*5*) that one-hundredth of a gram of glycerol covers 35.3 square meters of internal clay surfaces; thus a glycerol retention of 1 percent on internal surfaces corresponds to 35.3 m.²/g. Similar deductions indicate that a retention of 1 percent of glycerol on external surfaces corresponds to a specific surface of 17.65 m.²/g.

For the soils used in this study, the surface area value of the whole soil was computed by multiplying the surface area found for the passing 40-mesh fraction by the percentage of the whole soil which passes the 40-mesh sieve. The surface area of the particles coarser than 40 mesh is so small as to be negligible. A hypothetical example of these computations follows:

	Percent
Glycerol retention of passing 40-mesh fraction	3. 50
Glycerol retention of same after preliminary 600° C. heating	1. 50
Retention due to external surface	1. 50
Retention due to internal surface (3.50−1.50)	2. 00

The indicated surface area of the passing 40-mesh fraction was computed as follows:

External surface	1.50 x 17.65	=	26.5 m.²/g.
Internal surface	2.00 x 35.3	=	70.6 m.²/g.
Total			97.1 m.²/g.

The percentage of the whole soil passing the 40-mesh sieve was 65 percent. Thus, the surface area of the whole soil would be 97.1 m.²/g. x 0.65 = 63.1 m.²/g. For this study the figure 63.1 would be rounded to the nearest one-half square meter per gram, or 63 m.²/g.

Cement requirements

The cement requirement determinations were performed by the staff of the P. C. A. Soil Cement Laboratory, using the method described in their *Soil Cement Laboratory Handbook, 1956*. Briefly, this method is as follows:

(1) Determine the grain-size distribution and Atterberg limits of the soil.

(2) Determine the moisture-density relations of a mixture of the soil and an assumed percentage of cement.

(3) Mold durability test specimens at optimum moisture and at cement contents thought to bracket the cement requirement, and test through 12 cycles of freezing and thawing. (Wet-dry tests may also be made, but they were not used for the soils of this investigation.) For A–1, A–2–4, and A–2–5 soils, the cement requirement is specified by the P. C. A. as that cement content at which test specimens lose 14 percent of their weight during the 12 cycles and the accompanying brushing procedure. For A–2–6, A–2–7, A–4, and A–5 soils, the loss permitted is 10 percent; for A–6 and A–7 soils, it is 7 percent. These loss criteria are based on information from a great many laboratory tests, the performance of field projects, and outdoor exposure of several thousand specimens.

(4) Check the estimated cement factor by molding and testing small specimens for compressive strength to insure that adequate hardening takes place at this cement content.

(5) For reporting and for field use, the cement factor is converted from a weight basis to a volume basis by the use of the following relation:

$$\text{Percent cement by volume} = \frac{D - \frac{D}{C}}{94} \times 100$$

Where:

D = Oven-dry density of the soil-cement specimen in lb. per cu. ft.

C = 100 plus the percent cement by weight of the oven-dry soil, the quantity divided by 100.

(6) The final recommended cement content is based to some extent on the judgment of the testing engineer. For example, the cement content indicated by the durability test data might be in a critical range; that is, where a small decrease in cement content would lead to very much higher than allowable freeze-thaw losses. In such a case, inadequate mixing on the job could result in an unsatisfactory product, and to insure against this, the testing engineer would recommend a slightly higher overall cement content than that provided for by the durability test data.

Results and Discussion

Surface area values and cement requirements for the group A samples (Nos. S–32044 to S–32085) are listed in table 1. The surface areas range from 6 to 90 m.²/g. The cement requirement values quoted (in terms of percent by volume) are those actually recommended for construction by the P. C. A. laboratory. Analysis shows that a statistically significant correlation [4] exists, $r = 0.77$, significant at the 0.1 percent level. The following regression equation was derived from the data: Cement requirement = 0.06 (surface area) + 6.5. The standard error of estimate from this equation is 1.37 percent cement by volume. A plot of the relation is given in figure 1.

The statistical significance level of the correlation coefficient (0.1 percent) clearly indicates that a correlation actually exists. Nevertheless, the degree of correlation indicated by the correlation coefficient (0.77) is not strong enough to permit accurate predictions of cement requirement directly from surface area measurements.

Assuming the validity of the hypothesis that the surface area should be intimately associated with the cement need, the following possible reasons for not obtaining the expected closer correlation could be deduced:

(1) The surface area values used are based on the weight of the soil, surface area being expressed in square meters per gram, whereas the cement requirements are expressed on a volume basis. This naturally would weaken the correlation because there can be no overall relation between values expressed by weight and those expressed by volume, due to the variation in the densities of the different soil-cement products.

(2) As noted earlier, the cement requirement determined by the P. C. A. laboratory is based on the cement content at which maximum losses of 7, 10, or 14 percent occur during the 12-cycle freeze-thaw test; the limit applying depends on the AASHO classification of the soil. Two soils containing different clay minerals might have identical surface areas, but have greatly different grain-size distributions and plasticity indexes, thereby falling into different classifications. Accordingly, the cement requirement of one soil might be based on the 7-percent loss limit and that of the other on the 14-percent loss limit, despite their identical surface areas. In effect, this inserts a bias in the cement requirement values which is not correspondingly reflected in the surface area, and so tends to weaken the correlation.

(3) The cement requirements recommended are not derived solely from freeze-thaw loss data, but, as noted earlier, may be modified somewhat by engineering judgment. This added factor, while perfectly justified from a practical standpoint, is not related to a phys-

[4] See appendix on page 69 for definitions of statistical terms.

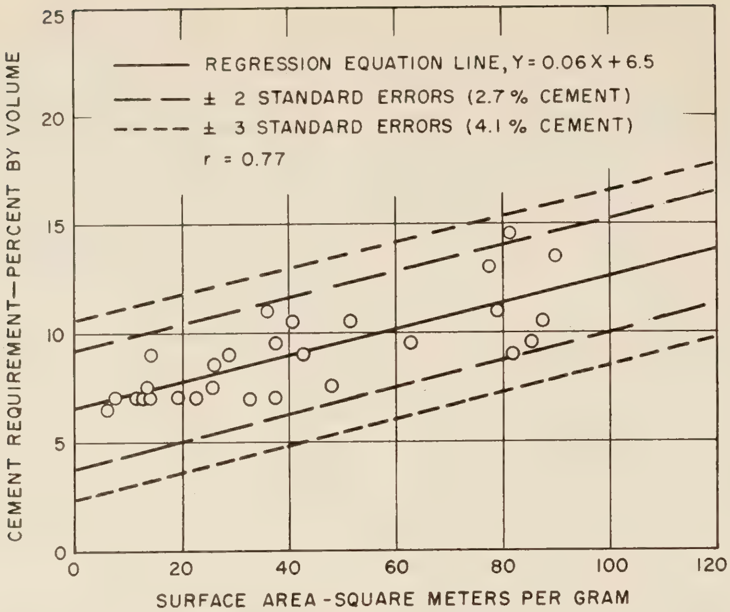

Figure 1.—Cement requirement recommended by the Portland Cement Association related to the surface area of group A soils.

ical measurement such as surface area and thus also tends to weaken the correlation.

(4) The cement requirement data are rounded to the nearest one-half percent of cement. Since the range in cement requirements encountered with these soils (6.5 to 14.5 percent) is only 16 times this figure, the rounding, while again justifiable from the practical standpoint, tends to weaken the correlation.

Cement requirement at 10-percent loss

It was thought that expressing the cement requirement in a different way would obviate these difficulties. Strictly for purposes of correlation, in place of the recommended cement requirement, it is proposed to use for all soils that cement content (in percent by weight) at which a 10-percent loss occurs in the freeze-thaw test. This places the cement content on a weight basis as is the surface area. The procedure eliminates the bias due to different limits of allowable loss for different soil groups by placing all of the soils on a uniform basis of 10-percent loss; it eliminates personal judgment factors; and when the cement contents are expressed to the nearest

Table 2.—Test data for 12 cycles of freezing and thawing, and data derived therefrom

B. P. R. soil No.	Test data furnished by the Portland Cement Association								Derived data			
	Specimen 1		Specimen 2		Specimen 3		Specimen 4		Cement content at 10-percent loss	Loss allowed in freeze-thaw test	Cement content at which allowed loss occurs	
	Cement content	Loss in weight	Cement content	Loss in weight	Cement content	Loss in weight	Cement content	Loss in weight			Percent by weight	Percent by volume
Group A Soils												
	Pct. by weight	*Pct.*	*Pct. by weight*	*Pct.*	*Pct. by weight*	*Pct.*	*Pct. by weight*	*Pct.*	*Pct. by weight*	*Pct.*		
S-32044	5	13	7	5	--------	--------	--------	--------	5.8	14	4.8	5.7
S-32045	10	13	12	10	14	6	--------	--------	12.0	7	13.5	13.2
S-32046	6	13	8	9	10	7	--------	--------	7.5	7	10.0	10.4
S-32047	8	10	10	5	12	4	--------	--------	8 0	7	9.2	9.7
S-32050	9	48	11	42	13	33	15	30	([1])	10	([1])	([1])
S-32051	9	36	11	8	13	5	--------	--------	10.8	10	10.8	10.9
S-32053	7	23	9	2	--------	--------	--------	--------	8.2	10	8.2	8.8
S-32055	5	13	7	8	--------	--------	--------	--------	6.0	14	[2] 4.5	[2] 5.4
S-32056	8	35	10	7	12	6	14	5	9.6	10	9.6	10.5
S-32060	3	18	5	6	7	1	--------	--------	4.3	14	3.6	5.0
S-32061	5	3	7	2	9	2	--------	--------	([1])	14	([1])	([1])
S-32062	12	25	14	14	16	4	--------	--------	14.8	7	15.3	13.7
S-32063	8	13	10	10	12	8	--------	--------	10.0	7	[2] 12.9	[2] 12.6
S-32064	3	20	5	2	--------	--------	--------	--------	4.0	14	3.6	4.8
S-32065	4	100	6	11	8	7	--------	--------	6.5	14	5.9	7.0
S-32066	8	6	10	5	--------	--------	--------	--------	([1])	10	([1])	([1])
S-32067	10	7	12	6	14	5	--------	--------	([1])	10	([1])	([1])
S-32068	5	20	7	15	--------	--------	--------	--------	([1])	14	[2] 7.4	[2] 8.7
S-32069	6	7	8	5	--------	--------	--------	--------	([1])	10	([1])	([1])
S-32070	4	18	6	9	8	5	--------	--------	5.7	10	5.7	6.8
S-32071	7	13	9	5	11	4	--------	--------	7.7	10	7.7	8.8
S-32072	3	18	5	9	--------	--------	--------	--------	4 9	10	4 9	6.3
S-32074	6	5	8	3	10	3	--------	--------	([1])	10	([1])	([1])
S-32075	10	10	12	6	14	5	--------	--------	10.0	10	10.0	10.3
S-32076	3	17	5	6	--------	--------	--------	--------	4 3	14	3.4	4.8
S-32078	4	24	6	13	--------	--------	--------	--------	([1])	14	5.8	7.0
S-32082	7	11	9	7	11	6	--------	--------	7.5	10	7.5	8.1
S-32083	7	19	9	10	11	6	--------	--------	9.0	10	9.0	9.5
S-32085	5	11	7	7	--------	--------	--------	--------	5.7	10	5.7	6.8
Group B Soils												
S-32572	7	44	9	9	--------	--------	--------	--------	8.9	14	8.6	10.4
S-32573	11	13	14	8	17	5	--------	--------	12.8	7	15.0	13.9
S-32575	13	22	15	13	17	4	--------	--------	15.6	10	15.6	15.0
S-32576	8	8	10	6	12	5	--------	--------	([1])	7	9.0	9.5
S-32577	6	10	8	8	10	5	--------	--------	6.0	10	6.0	6.8
S-32578	4	21	6	14	8	6	--------	--------	7.0	14	6.0	7.2
S-32579	8	12	10	8	--------	--------	--------	--------	9.0	10	9.0	10.1
S-32582	13	7	16	4	19	2	--------	--------	([1])	10	([1])	([1])
S-32583	11	12	14	4	17	2	--------	--------	11.7	10	11.7	11.3
S-32584	3	28	5	4	--------	--------	--------	--------	4.5	14	4.2	5.5

[1] Not available from data supplied. [2] Obtained by extrapolation.

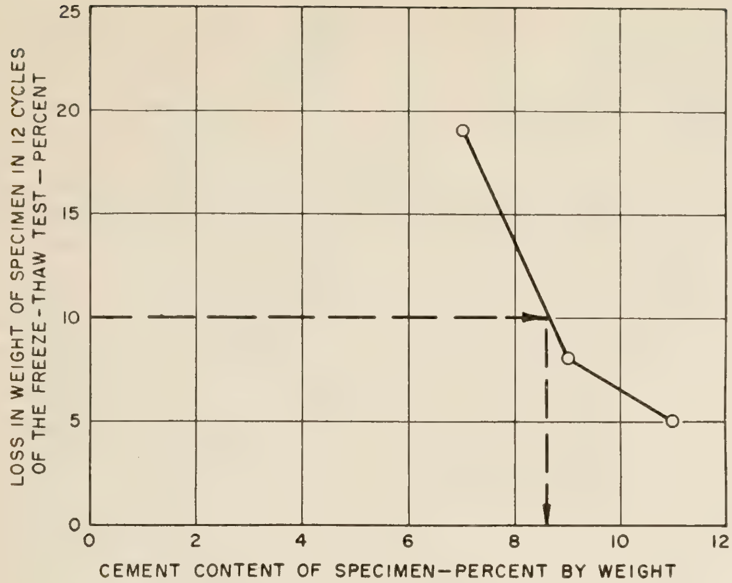

Figure 2.—Method of determining cement content at which a 10-percent loss would occur in a 12-cycle freeze-thaw test. (In this illustration, the required value is 8.6 percent.)

).1 percent, it eliminates bias due to excessive ·ounding of the values.

This expression of cement requirement was)btained by the procedure illustrated in igure 2. For the several freeze-thaw speci- nens prepared from the same soil at various :ement contents, a plot was made relating the ıctual test loss to cement content, and the)oints were connected by straight lines. The :ement content at which a 10-percent loss vould occur was then read directly from this)lot, by interpolation if necessary. The actual oss data and corresponding cement contents 'or all soils are given in table 2.

The relation between this new cement 'actor and surface area was investigated, with ·esults as shown in figure 3. The analysis ;howed a distinct and marked improvement n the degree of correlation existing, the cor- ·elation coefficient r being 0.94, which dem-)nstrates both the validity of the original ıypothesis that cement requirement and ;urface area are closely related, and also that :he disturbing factors previously discussed ıave been largely avoided by this method)f estimating the required cement content.

The regression equation appropriate for ;redicting the cement content by weight at which a 10-percent loss occurs was calculated ıs $Y = 0.076$ (surface area) $+ 3.93$. The stand- ırd error of estimate from this equation is).74 percent cement. It should be noted that for several of the samples, the available freeze-thaw test data could not be used to)btain the test value of the cement content at which a 10-percent loss would occur, except by questionable extrapolation; these samples have accordingly not been included in this correlation. In addition, three samples for which data were available (Nos. S–32056, S–32062, and S–32082) did not fit the correlation. Since these were more than three standard errors from the regression equation line, it is statistically valid to discard them from consideration on the grounds that they presumably do not belong to the same statistical population as the remainder of the samples. These three samples will be discussed later.

A sufficiently close correlation has thus been established to permit prediction of the cement content at which a 10-percent loss occurs from surface area measurements. It now remains to develop a procedure to convert the cement content, so predicted for an individual soil, back to an estimated cement requirement based on the specific freeze-thaw loss allowable for soils of its class. For those soils where a maximum of 10-percent loss is allowed (A–2–6, A–2–7, A–4, and A–5 soils), no adjustment of the predicted value is of course necessary; for those soils (A–6, A–7) having a maximum allowable loss of 7 percent, the predicted cement content would be increased; similarly for soils (A–1, A–2–4, A–2–5) where loss up to 14 percent is allowed, the predicted cement content would be decreased. By examination of the loss data, the appropriate corrections were estimated to be $+2.0$ percent and -0.7 percent, respectively. The corrected cement contents by weight can then be converted to a predicted cement requirement by volume through use of the formula previously listed.

The procedure was followed to obtain predicted cement requirements in percent by volume, which were then compared with the cement requirement (by volume) computed directly from the freeze-thaw test results. Agreement for group A samples (Nos. S–32044 through S–32085) was only reasonably good. The coefficient of correlation between the predicted and the test cement requirements was $r = 0.87$, and the standard error of estimate was 1.3 percent cement.

Summarizing the results to this point, it has been shown that the following conditions prevailed:

(1) A definite though not very precise correlation exists between surface area and the cement requirement (percent cement by volume) actually recommended by the P. C. A. for this first group of plastic soils.

(2) The correlation was greatly improved by using, instead of the recommended cement requirement by volume, the cement content

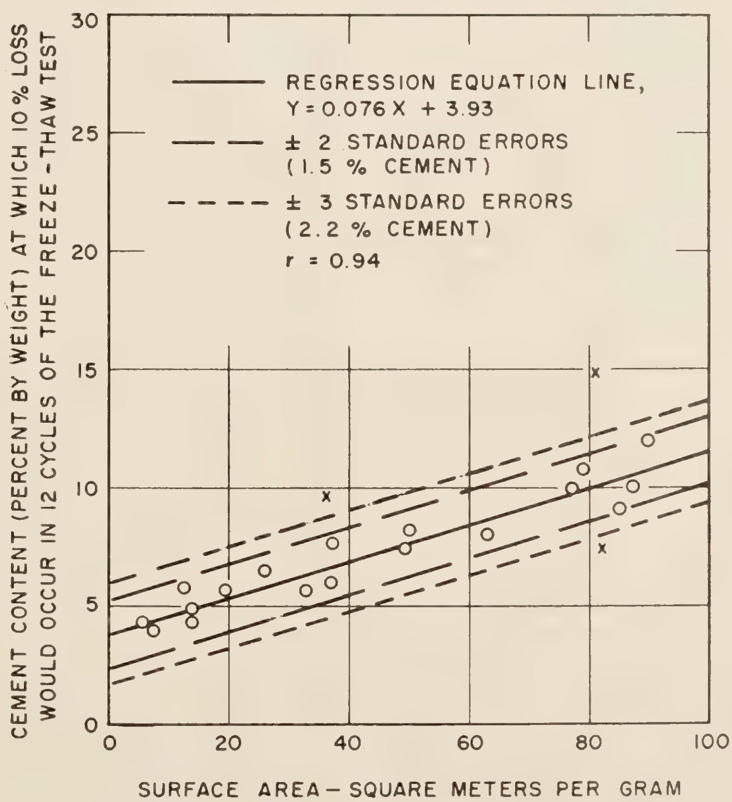

Figure 3.—Cement content at which a 10-percent loss would occur in a 12-cycle freeze-thaw test related to the surface area of group A soils.

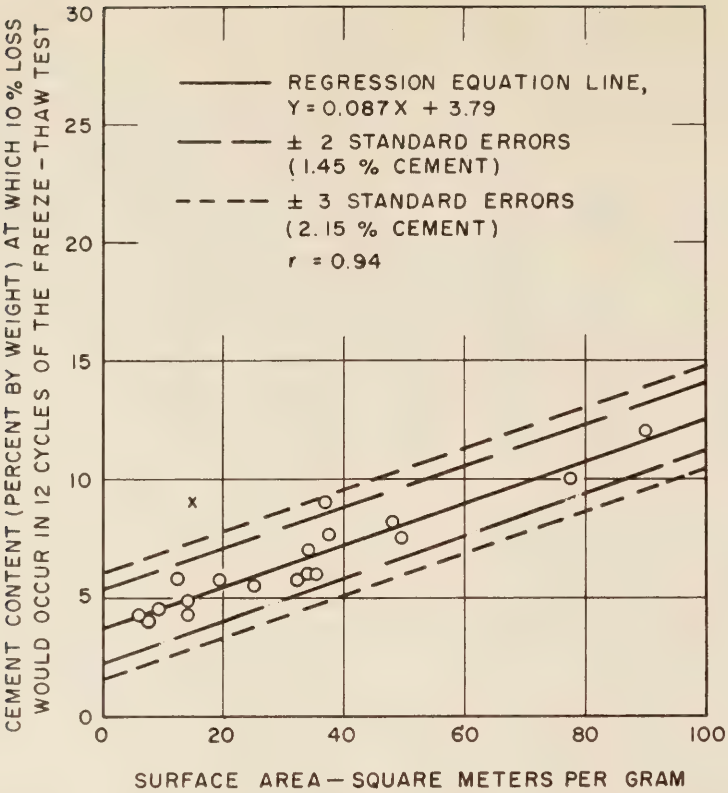

Figure 4.—Cement content at which a 10-percent loss would occur in a 12-cycle freeze-thaw test related to the surface area of soils having less than 45 percent silt.

by weight at which an arbitrary figure of 10-percent loss would occur in 12 cycles of the freeze-thaw test. The computed regression equation permitted satisfactory predictions of this value to be made from the surface area.

(3) A procedure was developed for correcting this cement factor to predict the cement content (by weight) at which a 7- or 14-percent loss would occur, and by use of a given formula involving the density of the soil-cement product, this prediction could be converted to a volume basis.

(4) Predictions thus made were in reasonable agreement with cement requirements derived directly from the freeze-thaw test data, the correlation between the two sets of values being 0.87.

Another group of soils (sample Nos. S–32572 through S–32584), received after the work on the first group had been completed, provided an opportunity to check the validity of these results. These samples are described in table 1 as group B, and cement requirement test data for them are listed in table 2. After the surface areas were determined for these soils, predictions of cement requirement (by volume) were computed, using the regression equation derived for the soils of group A and the additional procedure just outlined. A comparison of these predicted values with cement requirements computed directly from the freeze-thaw test data indicated good agreement for some of the samples but considerable deviations for others.

Upon examination of the engineering test data for group B soils, it was noted that all but one of the deviant samples were very high in silt content, silt being taken as that portion passing the 200-mesh sieve and coarser than 0.005 mm. When the data for the samples of group A were reexamined, it was found that here also high silt content was associated with relatively poor agreement between predicted and test results. In particular, it was noted that the three soils which were discarded from the previous correlation were high in silt content. For such soils, the cement requirement is evidently governed by some property or properties other than surface area.

Influence of silt content

In order to verify this premise, the data for both groups of soils taken together were divided into two categories on the basis of silt content. Upon examination of the data, an appropriate dividing line appeared to be at a silt content of 45 percent. The correlation between surface area and the cement content (by weight) at which a 10-percent loss occurred in the freeze-thaw test was recomputed separately for each category. The category (22 samples) with silt content under 45 percent showed a high degree of correlation ($r=0.94$) similar to that previously determined for the first batch alone.

One deviant sample (No. 32572) did not fit the correlation, and since it was considerably more than three standard errors from the regression equation line, it was discarded. This sample is discussed later. On the other hand, the category consisting of soils with silt contents of 45 percent or higher showed essentially zero correlation ($r=-0.03$). The data for the two categories of samples are plotted separately in figures 4 and 5. The striking difference in correlation coefficients demonstrates clearly that the two sets of samples represent different populations. For the soils of lower silt content, the cement requirement is essentially a function of surface area, as has previously been determined; but for soils of higher silt content, there is little relation between the two.

The regression equation was then calculated for the soils of low silt content. It differs only slightly from the corresponding equation previously calculated for group A samples alone. The new equation for cement content (by weight) at which a 10-percent loss occurs is: $y=0.087$ (surface area)$+3.79$. The equation and the previously described procedure were then used to predict the values of cement requirement for these soils. These predictions are compared in the upper part of table 3 with the cement requirements derived from the freeze-thaw test data. Agreement between the two sets of values is good (correlation coefficient $r=0.92$, standard error of estimate $=0.9$ percent cement), again with the exception of sample No. S–32572 previously noted as being a deviant sample. Neglecting this sample, the average deviation between predicted and test cement requirements was only 0.6 percent cement. Among the remaining

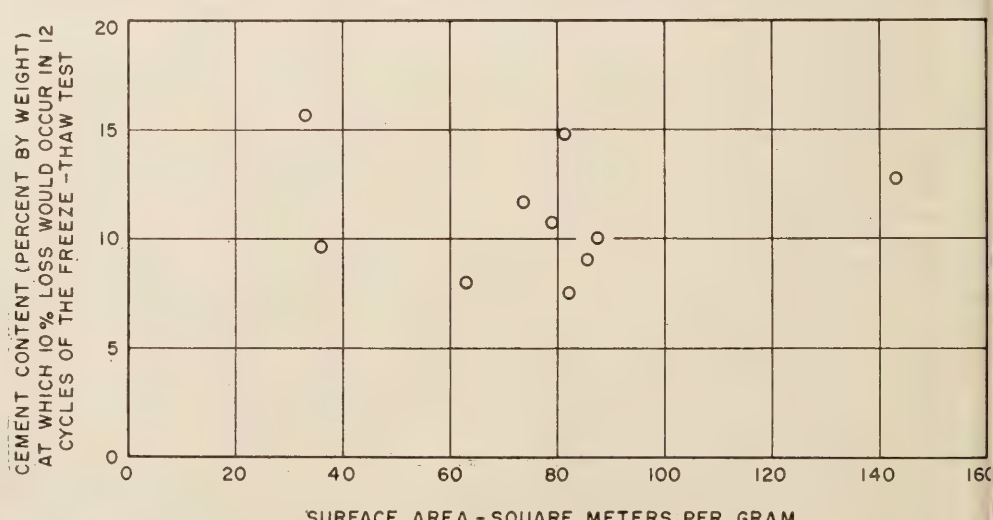

Figure 5.—Cement content at which a 10-percent loss would occur in a 12-cycle freeze-thaw test related to the surface area of soils having 45 percent or more silt.

soils, 6 were classified in the highly plastic ASHO groups A–2–6, A–2–7, A–6, and A–7. or all six soils, predictions were within 0.2 rcent cement of the test value.

ay mineral examination

The clay minerals present in the various mples were identified by X-ray diffraction chniques. The clay fractions were nearly l mixtures of two or more clay minerals, cluding among them montmorillonite, illite, olinite, chlorite, and vermiculite. Most ntained at least a detectable amount of ontmorillonite, and several were almost pure ontmorillonite. None of the clays was incipally illite or principally kaolinite, alough several consisted largely of a chloritee clay mineral. From the evidence availle, it seems that, in general, the cement quirement is influenced by the surface area elf, without regard to the specific type of ay mineral from which the surface area is rived.

No definite explanation has been found for e behavior of sample No. S–32572 (point X i figure 4), previously noted as a deviant mple. The actual cement requirement is uch higher than the predicted value. X-ray ffraction examination indicated that this soil nsists primarily of finely divided calcium rbonate (caliche), much of it in the clay e range. Further study is needed to deterine the effect of this type of material on the ment requirement.

From the results obtained, a procedure for edicting cement requirements of plastic soils less than 45 percent silt content was forulated. This is diagramed in figure 6. For ils to which it can be applied, the procedure minates the time-consuming freeze-thaw sts. It would still be necessary, however, prepare small specimens at and near the edicted cement requirement, and test them r compressive strength (or by other suitable eans) to insure that adequate hardening was tually taking place. Furthermore, the prected cement content should be modified by propriate engineering judgment to compente for factors such as difficulty of adequate ixing in the field and possible local variations i the soil materials.

It is recognized that these findings are based only a limited number of samples, and are t necessarily applicable to all soils. For ample, other authors have reported that rtain types of soil organic matter strongly fluence the cement requirement. Although number of the soils in this study were oderately high in organic matter, there was evidence of appreciable effects on cement quirement. With a larger group of samples, ils containing such deleterious organic mat-r might have been encountered.

Table 3.—Comparison of cement requirements predicted from surface area data with cement requirement from freeze-thaw test data

B. P. R. soil No.	AASHO classification	Surface area	Prediction of cement content at which 10-percent loss occurs [1]	Corrections for soils where 7- or 14-percent loss is allowed	Corrected prediction of cement requirement		Cement requirement computed from test data	Deviation (column 7 minus column 8)
					Percent by weight	Percent by volume		
Category 1: Plastic Soils With Less Than 45 Percent Silt Content								
		m.²/g.	*Percent by weight*	*Percent by weight*			*Percent by volume*	*Percent by volume*
S–32044	A–2–4(0)	12.5	4.9	–0.7	4.2	4.9	5.7	–0.8
S–32045	A–6 (7)	90	11.6	+2.0	13.6	13.1	13.2	–.1
S–32046	A–6 (2)	49.5	8.1	+2.0	10.1	10.6	10.4	+.2
S–32053	A–2–6 (1)	48	8.0	---------	8.0	8.6	8.8	–.2
S–32055	A–4 (8)	37.5	7.1	–.7	6.4	7.6	5.4	+2.2
S–32060	A–1–b (0)	14	5.0	–.7	4.3	5.9	5.0	+.9
S–32063	A–7–6 (14)	77.5	10.5	+2.0	12.5	12.4	12.6	–.2
S–32064	A–1–b (0)	75	4.4	–.7	3.7	5.0	4.8	+.2
S–32065	A–2–4 (0)	25.5	6.0	–.7	5.3	6.5	7.0	–.5
S–32068	A–2–4 (0)	28.5	6.3	–.7	5.6	6.8	8.7	–1.9
S–32070	A–4 (1)	19.5	5.5	---------	5.5	6.6	6.8	–.2
S–32071	A–4 (4)	37.5	7.1	---------	7.1	8.5	8.8	–.3
S–32072	A–2–6 (0)	14	5.0	---------	5.0	6.4	6.3	+.1
S–32076	A–2–4 (0)	6	4.3	–.7	3.6	4.8	4.8	.0
S–32078	A–2–4 (0)	20	5.5	–.7	4.8	5.8	7.0	–1.2
S–32085	A–4 (0)	32.5	6.6	---------	6.6	7.7	6.8	+.9
S–32576	A–6 (6)	36	6.9	+2.0	8.9	9.5	9.5	.0
S–32577	A 4 (1)	34	6.8	---------	6.8	7.7	6.8	+.9
S–32578	A–2–4 (0)	34	6.8	–.7	6.1	7.3	7.2	+.1
S–32579	A–4 (0)	37	7.0	---------	7.0	8.1	10.1	–2.0
S–32584	A–2–4 (0)	9	4.6	–.7	3.9	5.1	5.5	–.4
S–32572 [2]	A–2–4 (0)	14.5	5.1	–.7	4.4	5.6	10.4	–4.8
Category 2: Plastic Soils With 45 Percent or More Silt Content								
S–32047	A–6 (9)	63	9.3	+2.0	11.3	11.4	9.7	+1.7
S–32051	A–4 (8)	79	10.7	---------	10.7	10.8	10.9	–.1
S–32056	A–4 (8)	36	6.9	---------	6.9	7.7	10.5	–2.8
S–32062	A–7–6 (11)	81	10.8	+2.0	12.8	11.7	13.7	–2.0
S–32075	A–4 (8)	87.5	11.4	---------	11.4	11.6	10.3	+1.3
S–32082	A–4 (8)	82	10.9	---------	10.9	11.4	8.1	+3.3
S–32083	A–4 (8)	85.5	11.2	---------	11.2	11.6	9.5	+2.1
S–32573	A–7–6 (15)	143	16.2	+2.0	18.2	16.2	13.9	+2.3
S–32575	A–4 (8)	33	6.7	---------	6.7	7.1	15.0	–7.9
S–32583	A–4 (8)	73.5	10.2	---------	10.2	9.8	11.3	–1.5

[1] Determined by regression equation.
[2] Deviant sample discarded in computing the regression equation.

Appendix

The statistical terms employed in this study e as follows:

Correlation coefficient (*r*).—A term which dicates the degree of association or relation tween the measured values of one property and the corresponding measured values of another property, for a specified group of samples. This term varies from 1.0, indicating that a perfect functional relation exists and that one property could be predicted with absolute accuracy from knowledge of the other, to zero, which indicates a complete lack of relation between the two properties. If the relation is *direct*, i. e., if one property increases with an increase in the other, the correlation coefficient is *positive;* if the relation is *inverse* (one property decreases with an increase in the other), the correlation coefficient is *negative.* Generally, a correlation coefficient above 0.9 is required for the correlation to be good enough to permit predictions of one value from the other with a reasonable degree of accuracy.

Statistical significance level of correlation coefficient.—This is a measure of the probability that so large a correlation coefficient as has been computed from the data could arise by pure chance sampling from a population in which there is in fact no correlation. A 0.1-percent or even a 1-percent significance level indicates that a correlation almost certainly does exist.

Regression equation.—If a linear correlation exists between two properties of a group of samples, and a plot is made of property Y versus property X for all samples of the group, an array of scattered points results. A straight line may be drawn through the scattered points in such a way that it best fits the data, using as the criterion of "best fit" that the sum of the squares of the deviations of all of the points from the line is at a minimum. The equation of this line is called the regression equation, and its use permits the best estimate of values of property Y to be made from measured values of property X.

Standard error of estimate (Sy).—This is a measurement of deviation or degree of scatter of the points around the regression equation line. It has the same dimensions as the dependent variable, Y, and it provides an estimate of the uncertainty of the prediction of Y from X by means of the regression equation. If the normal distribution of errors holds, 19 out of 20 samples should fall within 2 standard errors of the regression equation line, and 997 out of 1,000 within 3 standard errors.

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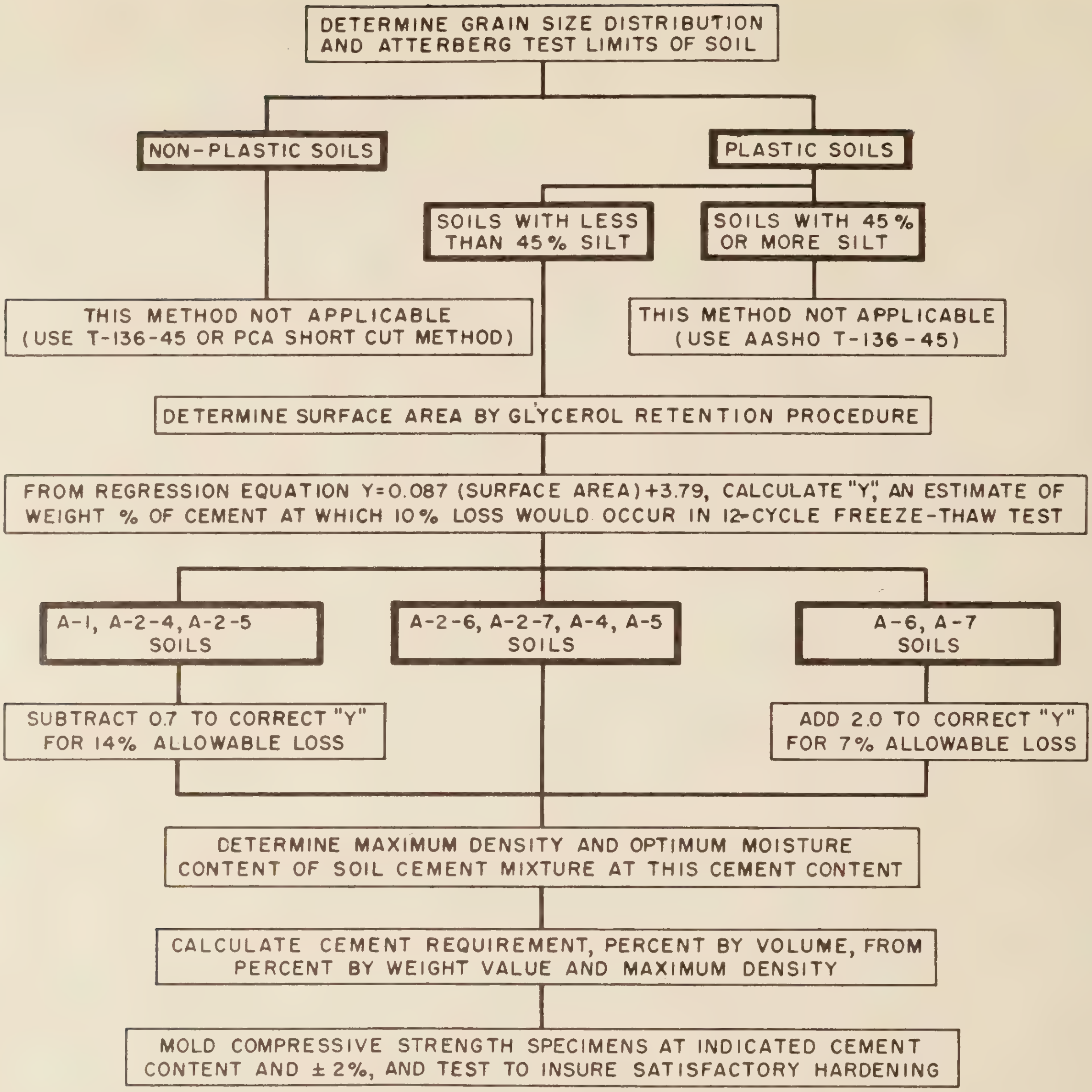

Figure 6.—Short-cut method, using surface area measurements, for determining cement requirement of plastic soils containing less than 45 percent silt.

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Lateral Placements of Trucks on Two-Lane Highways and Four-Lane Divided Highways

BY THE DIVISION OF TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported [1] by ASRIEL TARAGIN, Chief, Traffic Performance Branch

This article describes the driving habits of commercial-vehicle operators with respect to encroachment on the left traffic lane and on the shoulders of two-lane highways and four-lane divided highways. Lateral placement data were obtained at 119 rural locations in 17 States for 135,000 vehicles of all types of which more than 20,000 were commercial vehicles.

The positioning of vehicles in the traffic lane is particularly important to the highway engineer in designing the roadway to meet the requirements of the different classes of vehicles.

Both portland cement concrete and bituminous pavements were included in the study. Some pavement sections had grass or gravel shoulders; others had bituminous-paved shoulders; and still others, a combination of bituminous materials and gravel.

On two-lane highways with 12-foot traffic lanes and grass or gravel shoulders, only 0.8 percent of the trucks were encroaching on the shoulders, however, 2.3 percent were encroaching on the left traffic lane. The widest range in vehicle placements was found on road sections with matching bituminous pavements and shoulders. On these sections, nearly 70 percent of all trucks were traveling partly on the shoulders.

Studies made on level sections of four-lane divided highways indicated that placements in the right traffic lane were similar to those found on two-lane (two-directional) highways. On horizontal curves, drivers tended to travel closer to the center of the pavement on four-lane divided highways than was the case for two-lane highways.

THE Bureau of Public Roads has been conducting research studies for a number of years for the purpose of determining the effects of various highway geometric design features on driver behavior. The almost universally accepted width of 12 feet for a traffic lane was based largely on the results of a comprehensive study of the effect of roadway width on vehicle operations.[2] Although the results of this study were reported in 1945, the basic data were collected prior to World War II. At that time there was a very limited mileage of roads with 12-foot lanes. Since then, nearly all newly constructed main highways have 12-foot traffic lanes. Phenomenal increases in traffic volumes and particularly in the sizes and weights of commercial vehicles have also taken place during this period.

Results of recent road test studies [3] have highlighted the importance of the positioning of trucks in the traveled lane. Current data on lateral positions of trucks in normal traffic were needed for designing new highways and for scheduling vehicle operations on the AASHO Road Test in Illinois. The Bureau of Public Roads was requested to supply such data which were to be representative of two-lane highway and four-lane divided highway operations on sections with 12-foot traffic lanes. Bituminous and portland cement concrete pavements were to be included in the study.

Vehicle placement data were already available as a result of a number of studies recently conducted as part of the regularly scheduled driver behavior research program.[4] Special studies were conducted during the summer of 1956 in Illinois, Missouri, and Ohio to supplement existing data and to include several States in the general area of the AASHO Road Test. The results presented are based on placement data recorded at 119 rural locations in 17 States, as shown in table 1. A total of 135,000 vehicles of all types were studied, of which more than 20,000 commercial vehicles are included in the present analysis.

Commercial vehicles for this study include trucks with dual tires on the rear axles, tractor-trailer combinations, and buses. Placement data were recorded for a minimum of 100 commercial vehicles at each location. For the purpose of this article commercial vehicles will be identified as "trucks." These trucks constituted on the average about 15 percent of the total traffic. The range in

Table 1.—Number of locations and trucks studied for placements on pavement sections with 12-foot traffic lanes

State	Year of study	Two-lane highways			Four-lane divided highways		
		Locations studied	Trucks		Locations studied	Trucks	
			Number observed	Percent of total traffic		Number observed	Percent of total traffic
Arizona	1955	9	896	15	--------	--------	--------
California	1955	10	2,848	15	--------	--------	--------
Colorado	1955	3	547	10	--------	--------	--------
Idaho	1955	7	861	11	--------	--------	--------
Illinois	1956	6	1,119	19	10	1,532	12
Iowa	1955	3	622	15	--------	--------	--------
Louisiana	1956	5	917	17	1	150	6
Maryland	1954	5	1,192	13	--------	--------	--------
Missouri	1956	6	651	13	8	865	14
New Mexico	1955	2	472	15	--------	--------	--------
Ohio	1956	4	702	22	5	703	21
Oregon	1955	5	838	12	--------	--------	--------
Pennsylvania	1948	--------	--------	--------	13	2,615	14
Texas	1955	6	745	14	--------	--------	--------
Utah	1955	7	851	15	--------	--------	--------
Virginia	1953	--------	--------	--------	2	935	20
Washington	1955	2	342	10	--------	--------	--------
Total	----------	80	13,603	14	39	6,800	15

[1] Data were recorded in the field by William D. Whitby and Bernell A. Porter, engineering aids, Bureau of Public Roads, in cooperation with 17 State highway departments. Tabular and graphic material appearing in this issue were prepared under the direction of Robert E. Payne, statistical clerk, Bureau of Public Roads.

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Table 2.—Distribution of placements of trucks on rural two-lane pavements with 12-foot traffic lanes and grass or gravel shoulders

Distance from center of truck to centerline of pavement [1]	Distribution of placements on level tangent sections paved with—		Distribution of placements on level curve sections (2° to 6°) paved with—				All pavement sections
	Portland cement concrete	Bituminous materials	Portland cement concrete [2]		Bituminous materials [2]		
			Inside lane	Outside lane	Inside lane	Outside lane	
Feet	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
0.0–0.9	0.1	(3)	--------	--------	--------	--------	(3)
1.0–1.9	--------	0.2	--------	--------	0.5	0.2	0.2
2.0–2.9	.4	.4	--------	1.1	.2	1.7	.6
3.0–3.9	.7	1.2	2.1	--------	.4	4.2	1.5
4.0–4.9	6.5	11.6	16.6	9.1	4.7	20.3	11.5
5.0–5.9	28.2	30.8	31.2	33.6	18.8	39.7	30.4
6.0–6.9	47.2	41.8	38.5	47.4	50.4	27.8	42.2
7.0–7.9	15.5	11.1	11.6	8.8	24.0	5.6	12.8
8.0–8.9	1.4	1.8	--------	--------	.7	.5	.7
9.0–9.9	--------	.5	--------	--------	.3	--------	.1
10.0–10.9	--------	.1	--------	--------	--------	--------	(3)
11.0–11.9	--------	.3	--------	--------	--------	--------	(3)
12.0–12.9	--------	.2	--------	--------	--------	--------	(3)
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0

[1] For distances below 4 feet, trucks were straddling the centerline; for distances greater than 8 feet, trucks were encroaching on the shoulder.
[2] Inside lanes on curves to the right and outside lanes on curves to the left. [3] Less than 0.05 percent.

traffic volume was from 100 to 500 vehicles per hour for both directions of travel on two-lane roads, and from 200 to 1,000 vehicles per hour for one direction of travel on four-lane divided highways.

Truck placement data were recorded on bituminous and on portland cement concrete surfaces, on level tangent sections, and on horizontal curves of 2 to 6 degrees. There were sections with grass or gravel shoulders for all four of these conditions. In addition, there were bituminous-paved shoulders adjacent to the travel lane on some of the two-lane level tangent sections.

For the two-lane studies each truck was classified as to whether it was free-moving (uninfluenced by other traffic), meeting other vehicles (affected by opposing traffic), or other (neither free-moving nor meeting). Trucks overtaking and passing other vehicles were not included in any of these groups.

For the four-lane studies trucks were classified into three similar groups: The first group was free-moving trucks; the second group was identified as adjacent to other vehicles, in place of meeting other vehicles as used for two-lane roads; and the third group consisted of trucks that were not placed in either the first or second classification. It was found that because of the low volumes during the study periods the greatest percentage of trucks were in the free-moving group. The average lateral positions of the meeting trucks (on two-lane roads) and adjacent trucks (on four-lane roads) were 0.4 to 0.8 foot farther from the centerline of the pavement than the free-moving trucks. The distributions of placements for free-moving, for meeting or adjacent, and for other trucks have been combined in this article, and the results are shown in tables 2–5 and in figures 1 and 2.

The placement values shown in these tables and graphs are for the position of the center of the truck with respect to the center of th pavement. Placement data were recorded fo both the right and left wheels of each vehicle It was found, however, that the position of th center of the vehicle could be determined mor accurately and the results were simpler t analyze. The width of trucks for this stud was assumed to be 8 feet. Thus, to determin the position of the right edge of the righ wheels with respect to the pavement edge o 12-foot lanes, the value for the center of th truck was subtracted from 8. This value of is obtained by subtracting half the truck widt from the lane width, or 12 feet minus 4 feet For 11-foot lanes the factor is 7, and for 10 foot lanes the factor is 6.

Lateral Placements on Two-Lan Highways

Table 2 shows the distribution of placement of trucks on two-lane roads with grass or grave shoulders. Of the 40 locations studied, 3 were on level tangent sections and 8 were o horizontal curves. It is evident in figure

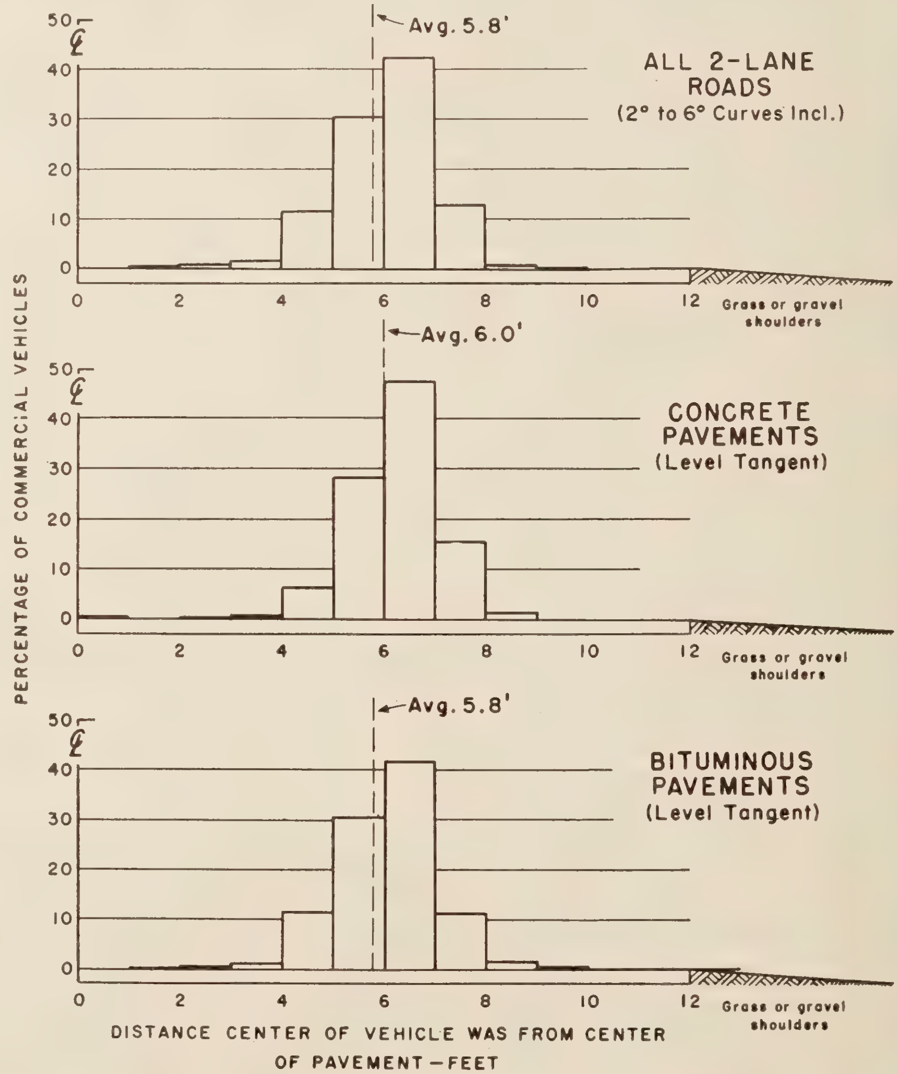


Figure 1.—Distribution of lateral positions of trucks on two-lane highways with grass or gravel shoulders.

able 3.—Placement data of trucks observed at individual locations on rural two-lane level tangent pavements with 12-foot bituminous traffic lanes and grass or gravel shoulders

State	Study section number	Shoulder width, in feet	Number of trucks observed	Average placement,[1] in feet	Percentage of trucks— Straddling center-line of pavement	Traveling entirely in traffic lane	Encroaching on shoulder
Iowa	1	2	90	5.5	--------	100.0	----------
Idaho	2	3	48	5.8	2.1	95.8	2.1
Colorado	3	4	210	5.6	2.9	93.3	3.8
Illinois	4	4	219	6.0	.9	99.1	----------
Missouri	5	4	107	5.7	.9	98.1	1.0
Arizona	6	4–6	165	6.1	--------	99.4	.6
Idaho	7	5	129	5.9	1.5	87.7	10.8
Missouri	8	5	119	5.6	--------	100.0	----------
Texas	9	5	181	6.0	--------	98.9	1.1
Maryland	10	5–7	254	5.8	1.2	98.8	----------
Louisiana	11	6	117	5.3	--------	98.3	1.7
Do	12	6	181	5.6	1.2	97.7	1.1
New Mexico	13	7	185	5.9	1.1	94.1	4.8
Louisiana	14	8	141	5.2	3.5	95.8	.7
Do	15	8	141	5.5	--------	100.0	----------
Maryland	16	8	289	5.7	7.2	92.8	----------
Do	17	8	259	6.1	.4	97.7	1.9
New Mexico	18	8	287	6.2	2.1	87.7	10.2
Utah	19	8	121	6.1	1.7	92.5	5.8
Colorado	20	10	170	5.9	1.8	90.5	7.7
Illinois	21	10	131	5.9	2.3	96.2	1.5
Do	22	10	211	5.9	1.9	95.4	2.7
Iowa	23	10	216	6.4	.5	92.6	6.9
Oregon	24	10	111	6.0	4.5	87.4	8.1
Washington	25	10	123	5.4	7.3	92.7	----------
Average or total	------	7	4,206	5.8	1.8	95.3	2.9

[1] Distance from center of truck to centerline of pavement.

hat on level tangent sections there was little ifference between the average placement on ituminous surfaces and concrete surfaces. 'he slightly greater range in the distribution of lacements on the bituminous pavements is due primarily to the larger sample size and the greater number of locations studied.

Horizontal curves of less than 6 degrees apparently did not affect the lateral placement of vehicles. This was particularly true on concrete surfaces. Because of the small difference between the placements on the several types of alinement on two-lane roads, the data for all sections were combined and are shown in the last column of table 2 and in the top graph of figure 1. Only 0.8 percent of all trucks encroached on the grass or gravel shoulders of two-lane roads with 12-foot traffic lanes. Encroachment on the left lane amounted to 2.3 percent. Nearly three-fourths of the trucks maintained a lateral position within 1 foot of the center of the lane. The average lateral position of the centers of all trucks was 5.8 feet from the centerline of the pavement on two-lane roads. In other words, the center of the average truck was 0.2 foot to the left of the center of the traffic lane.

Table 3 shows data for each of the 25 locations included in the results shown in the third column of table 2. At only 5 of the 25 locations did the lateral placements differ more than 0.3 foot from the average placement for all locations. Forty percent of the locations had placements within 0.1 foot of the average. It is obvious that the average data are very consistent and may be used with confidence.

Table 4 shows the distribution of placements on two-lane highways having 12-foot lanes with bituminous shoulders. Only level tangent sections are included. The widest range in the distribution of placements was found on sections with matching bituminous

'able 4.—Distribution of placements of trucks on rural two-lane level tangent pavements with 12-foot traffic lanes and bituminous-paved shoulders

Distance from center of truck to centerline of pavement [1]	Portland cement concrete and 8- to 10-foot bituminous shoulders	Bituminous-paved traffic lanes with— Combination-type shoulders, 8 to 10 feet wide [2]	Contrasting bituminous shoulders, 6 to 10 feet wide	Matching bituminous shoulders, 8 feet wide
Feet	*Percent*	*Percent*	*Percent*	*Percent*
0.0–0.9	-----	0.1	0.2	0.1
1.0–1.9	1.6	2.8	.1	.1
2.0–2.9	.1	.6	.2	.3
3.0–3.9	.1	.8	.7	.7
4.0–4.9	6.6	3.8	4.4	1.7
5.0–5.9	23.1	18.2	14.6	5.3
6.0–6.9	47.5	34.3	31.3	9.7
7.0–7.9	12.5	18.5	18.3	13.4
8.0–8.9	5.0	11.6	13.3	13.1
9.0–9.9	.8	3.5	6.1	12.0
10.0–10.9	.6	1.9	3.6	14.7
11.0–11.9	1.1	1.4	2.4	10.6
12.0–12.9	.8	2.4	2.4	7.6
13.0–13.9	.2	.1	2.0	5.7
14.0–14.9	-----	-----	.1	2.8
15.0–15.9	-----	-----	.2	.9
16.0–16.9	-----	-----	.1	.7
17.0 and over	-----	-----	-----	.6
Total	100.0	100.0	100.0	100.0

[1] For distances less than 4 feet, trucks were straddling the enterline of the pavement; for distances greater than 8 feet, rucks were encroaching on the shoulder.
[2] Four-foot contrasting bituminous shoulders plus 4 to 6 eet of gravel on the outside.

Table 5.—Distribution of placements of trucks on rural four-lane divided highways with 12-foot traffic lanes and grass or gravel shoulders

Distance from center of truck to centerline of pavement [1]	Distribution of placements on level tangent sections paved with— Portland cement concrete	Bituminous materials	Distribution of placements on level curve sections (2° to 6°) paved with— Portland cement concrete: Curve to right	Portland cement concrete: Curve to left	Bituminous materials, curve to left	All pavement sections
LEFT TRAFFIC LANE						
Feet	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
8.0–8.9	0.1	0.1	----------	----------	----------	(2)
7.0–7.9	.5	1.0	0.2	0.7	1.9	0.9
6.0–6.9	2.1	3.2	1.6	3.6	3.1	2.7
5.0–5.9	1.5	.9	1.0	.9	1.6	1.2
4.0–4.9	.7	.4	.3	1.6	.8	.8
3.0–3.9	.3	.1	.5	1.0	.4	.5
2.0–2.9	.3	----------	----------	.6	.8	.2
1.0–1.9	.1	.2	.2	.3	.4	.3
0.0–0.9	.1	----------	.4	1.1	----------	.3
Total	5.7	5.9	4.2	9.8	9.0	6.9
RIGHT TRAFFIC LANE						
0.0–0.9	0.3	----------	----------	0.3	----------	0.1
1.0–1.9	.2	0.4	0.7	1.5	2.4	1.1
2.0–2.9	.5	----------	.8	5.3	2.2	1.8
3.0–3.9	2.2	1.1	11.9	9.1	8.7	6.6
4.0–4.9	13.1	6.4	22.8	29.7	19.2	18.2
5.0–5.9	37.4	24.8	40.9	30.2	29.6	32.6
6.0–6.9	36.2	49.3	17.2	12.2	23.1	27.6
7.0–7.9	4.2	11.3	1.5	1.9	5.8	4.9
8.0–8.9	.2	.6	----------	----------	----------	.2
9.0–9.9	----------	.2	----------	----------	----------	(2)
Total	94.3	94.1	95.8	90.2	91.0	93.1

[1] For distances below 4 feet, trucks were straddling the centerline; for distances greater than 8 feet, trucks were encroaching on the shoulders for right lane and median for left lane.
[2] Less than 0.05 percent.

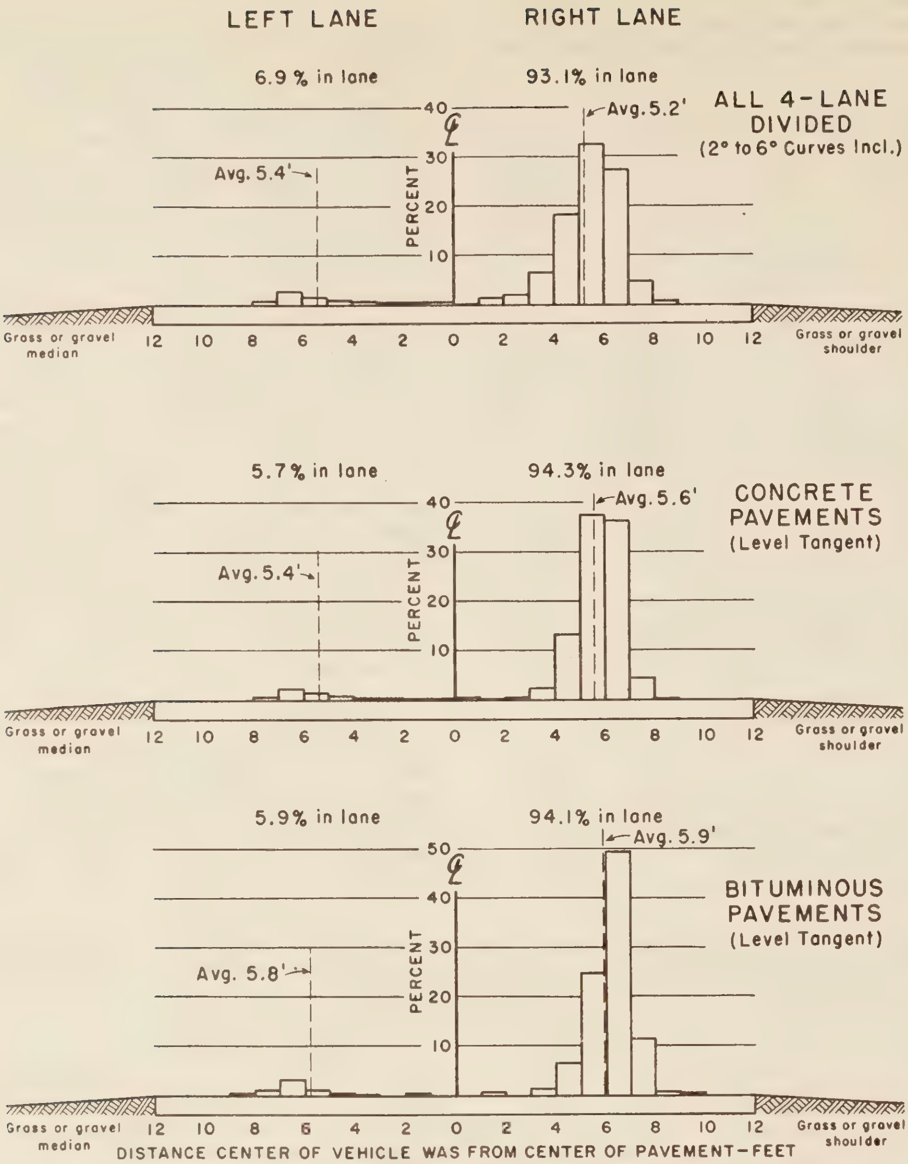


Figure 2.—Distribution of lateral positions of trucks on four-lane divided highways with grass or gravel shoulders and median.

shoulders and traffic lanes. On these section which did not have edge markings, nearl 70 percent of all trucks traveled partly on th shoulder. Of the trucks meeting other ve hicles (not shown in table 4), nearly 8 percent encroached on the shoulder. En croachment on the left traffic lane varied fro 1.2 to 4.3 percent. The greatest encroach ment on the left lane occurred on bituminou pavements with combination-type shoulder (part bituminous and part gravel), 8 to 1 feet in width, whereas the least encroachmen was found on sections with bituminous-pave traffic lanes and shoulders.

Lateral Placements on Four-Lane Highways

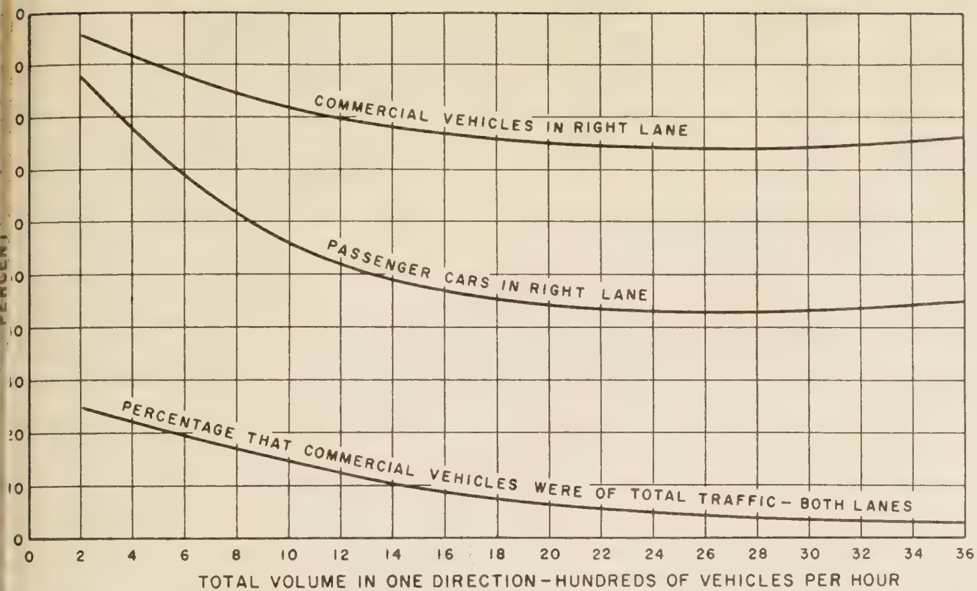
Table 5 and figure 2 show truck placement in the left and right lanes of four-lane divide highways. Of a total of 39 locations studied 27 locations were on level tangent section paved with portland cement concrete. Th placements in the right lane on level section of four-lane divided highways were ver similar to the placements on two-lane high ways. On horizontal curves, the placement on the four-lane divided highways were close to the center of the pavement than on th two-lane roads. Because of the low traffi volume, only about 7 percent of the truck traveled in the left traffic lane of divide highways. On the level tangent sections, th lateral position of trucks in the left lane wit respect to the center of the pavement wa about the same as the lateral position in th right lane.

Figure 3 shows the relation between th hourly volumes and the percentages of vehicle traveling in the right lane on a four-lan divided highway. At a volume of 500 or les vehicles per hour, at least 90 percent of th commercial vehicles and 73 percent of th passenger cars used the right lane. The per centage of trucks using the right lane decrease to 75 percent with an increase in the volum to 2,000 vehicles per hour. Above this volum

Table 6.—Number of locations, size of sample, average speeds, and average placements of trucks included in studies on two-lane highway and four-lane divided highways

Surface types of pavements with grass or gravel shoulders	Two-lane highways				Four-lane divided highways						
						Left traffic lane			Right traffic lane		
	Number of locations	Size of sample	Average speed	Average placement [1]	Number of locations	Size of sample	Average speed	Average placement [1]	Size of sample	Average speed	Average placement [1]
Level Tangent Sections											
			M. p. h.	*Ft.*			*M. p. h.*	*Ft.*		*M. p. h.*	*Ft.*
Portland cement concrete pavements	7	1,473	43.5	6.0	27	348	52.3	5.4	4,602	46.2	5.6
Bituminous pavements	25	4,206	46.8	5.8	3	49	47.4	5.8	585	44.1	5.9
Level Curves (2 to 6 degrees)											
Portland cement concrete pavements: [2]											
Curve to right	2	169	43.2	5.7	3	20	46.7	5.2	475	40.7	5.1
Curve to left		192	41.4	5.8	4	62	53.0	5.0	454	49.0	4.6
Bituminous pavements: [2]											
Curve to right	6	447	42.7	6.2	------	------	------	------	------	------	------
Curve to left		447	42.0	5.3	2	16	43.6	5.6	189	42.0	5.0
Summary											
All pavement sections	40	6,934	43.3	5.8	39	495	48.6	5.4	6,305	44.4	5.2

[1] Distance from center of truck to centerline of pavement. [2] For two-lane highways, "curve to the right" indicates an inside lane and "curve to the left" indicates an outside lane.



igure 3.—Percentage of vehicles using right-hand lane of a four-lane divided highway and the percentage of commercial vehicles in the traffic stream during various hourly traffic volumes.

ible 7.—Distribution of placements of trucks on rural two-lane level tangent pavements of different widths and shoulder surface types

Distance from center of truck to centerline of pavement [1]	Distribution of placements on 10-foot traffic lanes with—		Distribution of placements on 11-foot traffic lanes with—		Distribution of placements on 12-foot traffic lanes with—	
	Contrasting bituminous shoulders	Grass or gravel shoulders	Contrasting bituminous shoulders	Grass or gravel shoulders	Contrasting bituminous shoulders	Grass or gravel shoulders
Feet	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
0.0–0.9	----------	0.2	0.2	0.2	0.2	0.1
1.0–1.9	0.5	.2	.2	.2	.1	.1
2.0–2.9	1.0	.8	.1	1.2	.2	.4
3.0–3.9	4.9	3.3	1.9	5.0	.7	.9
4.0–4.9	30.8	32.6	7.8	19.1	4.4	9.0
5.0–5.9	23.8	52.2	30.0	41.2	14.6	29.5
6.0–6.9	14.1	9.6	37.8	30.3	31.3	44.5
7.0–7.9	13.1	.4	10.8	2.8	18.3	13.3
8.0–8.9	7.9	.1	4.0	(2)	13.3	1.6
9.0–9.9	3.9	.5	2.1	----------	6.1	.2
10.0–10.9	----------	.1	2.1	----------	3.6	.1
11.0–11.9	----------	----------	1.2	----------	2.4	.2
12.0–12.9	----------	----------	1.6	----------	2.4	.1
13.0–13.9	----------	----------	.2	----------	2.0	----------
14.0–14.9	----------	----------	----------	----------	.1	----------
15.0–15.9	----------	----------	----------	----------	.2	----------
16.0–16.9	----------	----------	----------	----------	.1	----------
Total	100.0	100.0	100.0	100.0	100.0	100.0

[1] For distances less than 4 feet, trucks were straddling the centerline of the pavement; for distances greater than 6, 7, and 'eet for 10-, 11-, and 12-foot traffic lanes, respectively, trucks were encroaching on the shoulder.
[2] Less than 0.05 percent.

able 8.—Number of locations, size of sample, average speeds, and average placements of trucks on two-lane highways with 10-, 11-, and 12-foot traffic lanes

Lane widths and shoulder types	Number of locations	Size of sample	Average speed	Average placement [1]
10-foot traffic lanes with—			*M. p. h.*	*Feet*
Contrasting bituminous shoulders	2	140	42.6	5.7
Grass or gravel shoulders	8	716	42.6	5.0
11-foot traffic lanes with—				
Contrasting bituminous shoulders	6	1,142	44.4	6.3
Grass or gravel shoulders	10	1,525	40.5	5.3
12-foot traffic lanes with—				
Contrasting bituminous shoulders	15	2,926	48.4	7.1
Grass or gravel shoulders	32	5,679	45.2	5.9

[1] Distance from center of truck to centerline of pavement.

the percentage of trucks in the right lane remained constant at 75 percent. In other words, the left lane of travel was used by less than 25 percent of the trucks during the heaviest traffic volumes, and by less than 10 percent of the trucks during volumes below 500 vehicles per hour.

Trucks constituted a decreasing proportion of the total traffic as the volume increased. At volumes of 500 vehicles per hour for one direction of travel on a four-lane divided highway, 20 percent of the vehicles were trucks; at volumes of 3,500 vehicles per hour only 3 percent of the vehicles were trucks. The results shown in figure 3 are based on recent data recorded on a principal four-lane divided highway south of Washington, D. C.

Table 6 is a summary of the studies made on both two-lane highways and four-lane divided highways. Included are the number of locations studied, the size of samples, the average speeds of vehicles, and the average placements with respect to the centerline of the pavements.

In addition to placement data on pavements with 12-foot lanes, there is interest in data on pavements with lane widths other than 12 feet. Table 7 shows the distribution of placements of trucks on two-lane roads with 10-, 11-, and 12-foot lanes including both bituminous and concrete pavements. For each lane width the data are shown separately for sections with bituminous shoulders and grass or gravel shoulders.

As indicated in table 8 for traffic lanes of the same width, the placements on sections with bituminous shoulders are considerably farther from the centerline of the pavement than on sections with grass or gravel shoulders. The results of this study confirm the results of earlier studies [5] that bituminous-paved shoulders which appear distinctly different from the traffic lane increase the effective pavement width at least 2 feet, regardless of the lane width.

[5] See footnote 2, p. 71.

Highway Statistics, 1956

The Bureau's Highway Statistics, 1956, the twelfth of the bulletin series presenting annual statistical and analytical tables of general interest on the subjects of motor fuel, motor vehicles, highway-user taxation, financing of highways, and highway mileage is now available.

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