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The triaxial compression test apparatus

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E. A. STROMBERG, Editor

Triaxial Compression Test Results Applied to Flexible Pavement Design

BY THE DIVISION OF PHYSICAL RESEARCH
PUBLIC ROADS ADMINISTRATION

Reported by E. S. BARBER, Highway Engineer

This discussion concerns several relations between triaxial compression test results and the thickness of flexible pavement based on stress distribution in elastic materials and the strength of plastic materials. Tables and graphs of stresses, displacements, and bearing capacity under normal and tangential surface loads are presented. Their application to the design of thickness of flexible pavement courses is outlined, using unit values for the components derived from triaxial compression tests. Designs based on allowable displacement, overstress at a point, and total failure are considered. The analyses are not proposed as determinative design procedures but are rather intended as aids in studying the behavior of pavements in service, and as guides in planning and conducting experimental studies of the interaction of pavements and subgrades.

THE SELECTION of thickness of flexible pavement (surface and base) required to support a given loading over a given subgrade is generally based on service records and observations of field performance of similar pavements constructed on similar subgrades. To evaluate the similarity of various pavement materials and subgrades, a method of testing the materials is necessary. In correlating observations of pavement performance involving various loads and materials, it is helpful to have a theoretical relation between the pavement thickness and the material test results.

Thus, while the results of field observations are required, their application may be broadened and their evaluation made more quantitative if they are correlated by means of test results on the component materials with the aid of theoretical relations. The following discussion concerns several relations between triaxial compression test results and pavement thickness based on stress distribution in elastic materials and the strength of plastic materials.[1]

ELASTIC STRESS DISTRIBUTION

The earth may be considered as an elastic mass, bounded only by the surface horizontal plane and hence semi-infinite in extent. When such a mass is subjected at the surface to a uniform vertical pressure over a circular area, every element in the mass is acted upon by the stresses shown in figure 1. Three planes are considered to intersect at the element: The horizontal plane, parallel to the surface; the radial plane, vertical and along the radial line between the load center and the element; and the tangential plane, vertical and perpendicular to the radial plane. Acting upon the element, in these three planes, are the normal stresses shown in figure 1: The tangential normal stress, p_t; the radial normal stress, p_r; and the vertical normal stress, p_z. Two equal shear stresses are developed: The vertical shear stress, s_z, acting in the vertical direction on the tangential plane; and the radial shear stress, s_r, acting in the radial direction on the horizontal plane. The tangential normal stress is a principal stress, and therefore there are no shear stresses in the plane on which it acts.

These stresses have been evaluated analytically by Love (*1*)[2] and tabulated numerically by Tufts (*2*). Some corrections were made to Tufts' tabulation by means of influence charts

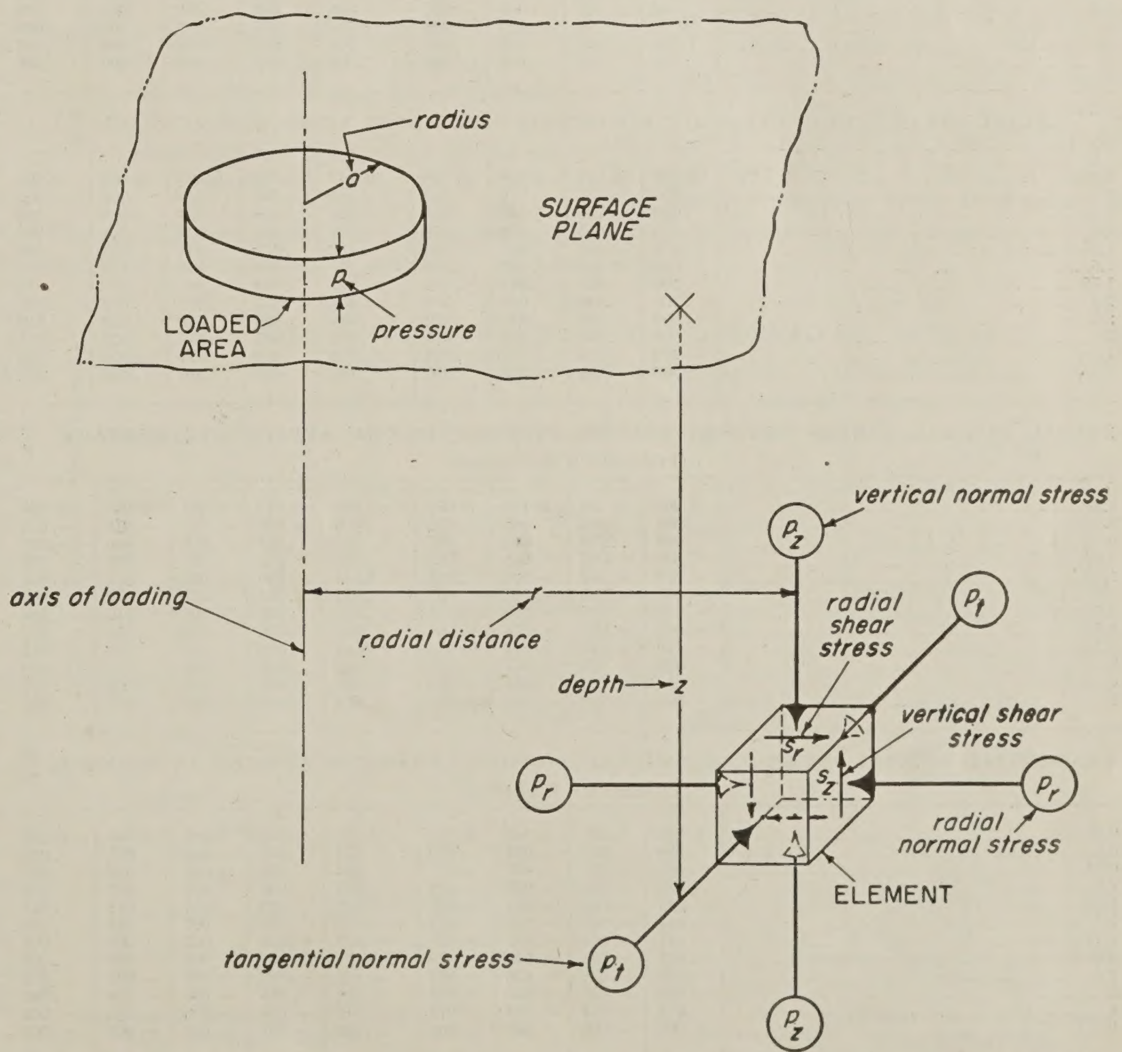

Figure 1.—Stresses on an element from pressure applied at the surface.

[1] A complete notation for this article appears on page 8.
[2] Italic numbers in parentheses refer to the bibliography, p. 8.

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Table 1.—*Ratio of stresses transmitted to a point in a semi-infinite mass from a surface load uniformly distributed over a circular area*

Depth of point÷radius, $\frac{z}{a}$	Horizontal radial distance÷radius, $\frac{r}{a}$: 0	0.25	0.5	1.0	1.5	2.0	2.5	3.0	4.0
VERTICAL NORMAL STRESS TRANSMITTED TO POINT÷PRESSURE APPLIED AT SURFACE, $\frac{p_z}{p}$									
0.25	0.986	0.983	0.964	0.460	0.015	0.002	0.000	0.000	0.000
0.5	.911	.895	.840	.418	.060	.010	.003	.000	.000
0.75	.784	.762	.691	.374	.105	.025	.010	.002	.000
1.0	.646	.625	.560	.335	.125	.043	.016	.007	.000
1.25	.524	.508	.455	.295	.135	.057	.023	.010	.001
1.5	.424	.413	.374	.256	.137	.064	.029	.013	.002
1.75	.346	.336	.309	.223	.135	.071	.037	.018	.004
2.0	.284	.277	.258	.194	.127	.073	.041	.022	.006
2.5	.200	.196	.186	.150	.109	.073	.044	.028	.011
3	.146	.143	.137	.117	.091	.066	.045	.031	.015
4	.087	.086	.083	.076	.061	.052	.041	.031	.018
5	.057	.057	.056	.052	.045	.039	.033	.027	.018
RADIAL NORMAL STRESS TRANSMITTED TO POINT÷PRESSURE APPLIED AT SURFACE, $\frac{p_r}{p}$ (POISSON'S RATIO=0.5)									
0.25	0.643	0.626	0.565	0.385	0.144	0.058	0.028	0.014	0.004
0.5	.374	.360	.325	.286	.196	.098	.050	.027	.008
0.75	.208	.204	.196	.209	.175	.112	.064	.044	.012
1.0	.116	.118	.123	.149	.146	.104	.069	.045	.022
1.25	.067	.072	.080	.107	.116	.096	.069	.047	.026
1.5	.040	.046	.055	.078	.091	.082	.064	.047	.026
1.75	.025	.028	.035	.056	.070	.068	.058	.046	.027
2.0	.016	.019	.024	.041	.053	.057	.052	.042	.027
2.5	.008	.009	.013	.023	.033	.038	.038	.035	.025
3	.004	.006	.008	.014	.021	.026	.028	.026	.022
4	.001	.002	.003	.006	.009	.012	.015	.016	.016
5	.001	.001	.002	.003	.005	.007	.008	.009	.010
TANGENTIAL NORMAL STRESS TRANSMITTED TO POINT÷PRESSURE APPLIED AT SURFACE, $\frac{p_t}{p}$ (POISSON'S RATIO=0.5)									
0.25	0.643	0.628	0.580	0.243	0.019	0.005	0.001	0.000	0.000
0.5	.374	.359	.317	.141	.028	.007	.003	.001	.000
0.75	.208	.197	.170	.085	.025	.008	.003	.001	.000
1.0	.116	.109	.096	.054	.021	.008	.003	.001	.000
1.25	.067	.063	.056	.035	.016	.007	.003	.001	.000
1.5	.040	.037	.034	.023	.012	.006	.003	.001	.000
1.75	.025	.024	.022	.015	.009	.005	.003	.001	.000
2.0	.016	.015	.014	.011	.007	.004	.002	.001	.000
2.5	.008	.007	.007	.006	.004	.003	.002	.001	.000
3	.004	.004	.003	.003	.002	.002	.001	.001	.000
4	.001	.001	.001	.001	.001	.001	.000	.000	.000
5	.001	.001	.000	.000	.000	.000	.000	.000	.000
RADIAL SHEAR STRESS TRANSMITTED TO POINT÷PRESSURE APPLIED AT SURFACE, $\frac{s_r}{p}$									
0.25	0.000	0.024	0.065	0.299	0.042	0.014	0.003	0.002	0.001
0.5	.000	.057	.129	.262	.102	.032	.013	.006	.002
0.75	.000	.069	.141	.221	.128	.053	.024	.013	.003
1.0	.000	.065	.124	.178	.128	.069	.033	.018	.007
1.25	.000	.053	.101	.146	.118	.072	.039	.023	.010
1.5	.000	.041	.080	.119	.104	.071	.045	.028	.012
1.75	.000	.033	.062	.094	.091	.068	.046	.030	.014
2.0	.000	.026	.048	.070	.078	.062	.045	.032	.015
2.5	.000	.016	.030	.050	.056	.050	.041	.032	.018
3	.000	.009	.019	.034	.040	.040	.035	.029	.018
4	.000	.005	.009	.018	.022	.024	.024	.022	.016
5	.000	.002	.005	.010	.013	.015	.016	.016	.013
RADIAL NORMAL STRESS TRANSMITTED TO POINT÷PRESSURE APPLIED AT SURFACE, $\frac{p_r}{p}$ (POISSON'S RATIO=0)									
0.25	0.265	0.248	0.199	0.082	[1] −0.026	−0.045	−0.061	−0.046	−0.026
0.5	.098	.087	.063	.075	.056	−.006	−.016	−.018	−.017
0.75	.008	.008	.009	.055	.067	.033	.009	.000	−.010
1.0	−.030	−.025	−.015	.033	.058	.041	.021	.008	.000
1.25	−.043	−.036	−.024	.018	.044	.040	.026	.015	.004
1.5	−.044	−.038	−.028	.007	.032	.035	.027	.019	.006
1.75	−.041	−.036	−.028	−.001	.022	.028	.025	.019	.009
2.0	−.037	−.033	−.024	−.006	.013	.022	.022	.019	.010
2.5	−.028	−.026	−.022	−.010	.004	.012	.016	.016	.011
3	−.022	−.020	−.017	−.010	−.002	.006	.010	.011	.010
4	−.014	−.012	−.011	−.009	−.004	−.001	.003	.005	.007
5	−.009	−.009	−.008	−.007	−.004	−.002	.000	.001	.004
TANGENTIAL NORMAL STRESS TRANSMITTED TO POINT÷PRESSURE APPLIED AT SURFACE, $\frac{p_t}{p}$ (POISSON'S RATIO=0)									
0.25	0.265	0.259	0.245	0.183	0.132	0.095	0.061	0.046	0.026
0.5	.098	.094	.085	.070	.074	.061	.048	.037	.022
0.75	.008	.007	.007	.017	.034	.038	.034	.029	.019
1.0	−.030	−.030	−.025	−.009	.011	.020	.023	.021	.016
1.25	−.043	−.042	−.037	−.021	−.002	.009	.014	.015	.013
1.5	−.044	−.043	−.039	−.025	−.010	.001	.007	.010	.010
1.75	−.041	−.040	−.037	−.026	−.013	−.003	.003	.006	.008
2.0	−.037	−.036	−.033	−.025	−.015	−.007	−.001	.003	.006
2.5	−.028	−.028	−.026	−.022	−.015	−.010	−.004	−.001	.002
3	−.022	−.021	−.020	−.018	−.014	−.010	−.006	−.004	.000
4	−.014	−.014	−.014	−.012	−.010	−.008	−.006	−.005	−.002
5	−.009	−.009	−.009	−.009	−.008	−.007	−.006	−.005	−.002

[1] Minus sign indicates tensile stress.

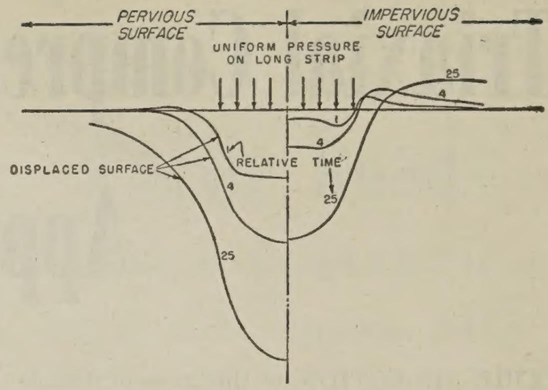


Figure 2.—Effect of time of consolidation on displacement of pervious and impervious loaded surfaces.

(*3*) to derive table 1, which shows the ratio of these stresses to the pressure applied at the surface, for various depths and for various radial distances measured from the center of the loaded area in terms of its radius. The vertical normal stress and the horizontal radial shear stress are independent of Poisson's ratio (the ratio of lateral to axial strain in a simple compression test). The radial and tangential normal stresses are given for Poisson's ratio of 0 and 0.5. Stresses for Poisson's ratio other than 0 and 0.5 may be obtained by direct interpolation: thus, the stress for Poisson's ratio of 0.25 is halfway between the tabulated values.

Integration of the stresses shown in table 1 gives the vertical displacement factors shown in table 2. While the stresses in a homogeneous mass are independent of the modulus of elasticity, the displacement is inversely proportional to the modulus. The displacement may be divided into two parts—that due to volume change only, corresponding to Poisson's ratio=0; and that due to lateral displacement at constant volume, corresponding to Poisson's ratio=0.5.

In comparing displacements measured in field tests with calculated values, consideration must be given to the fact that displacement due to volume change of wet soils is delayed because of the time required for the stress to be transferred to the soil solids as part of the water is forced out. This time effect and its dependence upon the permeability of the surface (*4*) is illustrated in figure 2. The curves

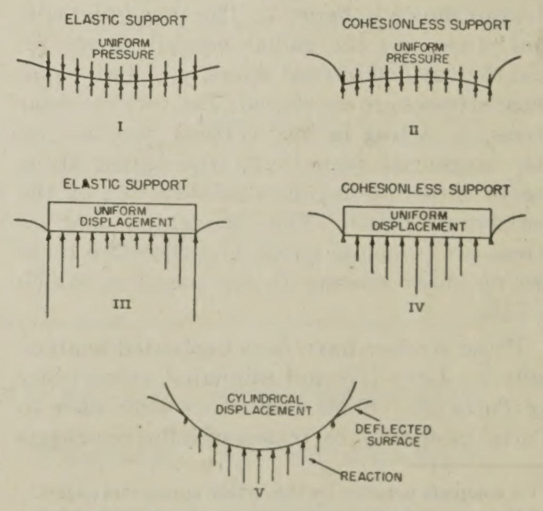


Figure 3.—Reactions under surface loads.

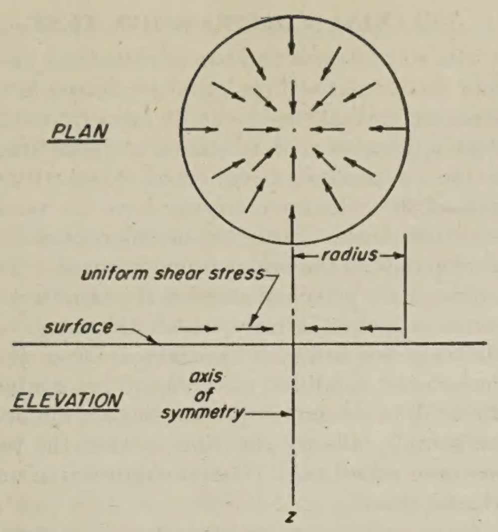


Figure 4.—Inward acting shear stresses produced by pneumatic tires.

on the left represent settlements when the surface is pervious, those on the right show the settlements at the same time when no drainage can occur at the surface. The relation between stress, volume change, and time for soil samples is determined in the consolidation test whereby a disk of soil is encircled with a metal ring and compressed between two pervious plates.

The time effect must also be considered when viscous materials are loaded. Under constant pressure, the surface displacement outside the loaded area may increase at first and then decrease as the displacement under the load continues to increase (*5*).

Figure 3, case I, shows the deflection of the surface of an elastic mass caused by a uniform pressure. If the supporting material is overstressed by a uniform pressure, the edge deflection increases and may exceed that at the center as indicated in figure 3, case II, for cohesionless support. If an elastic circular bearing block is placed between a uniform pressure and an elastic support, the reaction is concentrated toward the edge of the block (*6*). As the rigidity of the block increases, the edge stress increases without limit giving a uniform displacement (fig. 3, case III) equal to either 0.923 times the average displacement under a uniform pressure or 0.785 times the axial displacement under a uniform pressure (fig. 3, case I). Because of the lack of bearing capacity of unconfined cohesionless material, a rigid block on the surface of sand produces a reaction concentrated on the axis of loading (fig. 3, case IV).

The reaction under smooth-faced rollers and wheels without pneumatic tires approaches that shown in figure 3, case V, under a rigid solid cylinder. The average reaction is 0.785 times the maximum reaction (*7*).

The maximum reaction is

$$\sqrt{\frac{P'E}{R(1-\mu^2)}}$$

where: P' = load per unit length,
R = radius of loading cylinder,
E = modulus of elasticity of the elastic support,
μ = Poisson's ratio of the elastic support.

The reaction thus increases with increases in the load and modulus of elasticity, E, and decreases with increases in the radius, R.

Another factor which must be considered in evaluating loading tests is the production of inward acting shear stresses at the surface under pneumatic tires (*8*) as shown in figure 4. While measured shear stresses show a maximum at the quarter points of the diameter of the loaded area, their effect may be approximated by the vertical normal stress and displacement shown in table 3 for a uniformly applied shear stress (*9*). These normal stresses added to those due to the vertical applied pressures could account for the fact that Spangler (*10*) measured pressures transmitted to a strong base through a surface course which were greater than the inflation pressure of the tire used for applying the load. The shear stress also tends to increase the axial displacement more than that at the edge.

Table 2.—*Factors for calculation of displacement due to uniform pressure over a circular area* [1]

DISPLACEMENT FACTOR FOR POISSON'S RATIO=0.5

Depth of point÷radius, $\frac{z}{a}$	Horizontal radial distance÷radius, $\frac{r}{a}$										
	0	0.25	0.5	0.75	1.0	1.25	1.5	2	2.5	3	4
0	1.50	1.48	1.40	1.25	0.95	0.66	0.54	0.39	0.30	0.26	0.18
0.5	1.34	1.31	1.23	1.09	.89	.68	.55	.40	.31	.27	.19
1	1.06	1.05	.98	.89	.78	.66	.56	.41	.32	.27	.19
1.5	.83	.83	.79	.73	.67	.60	.52	.40	.32	.28	.20
2	.67	.67	.65	.62	.57	.53	.48	.39	.32	.28	.20
3	.47	.47	.46	.45	.43	.41	.38	.34	.30	.27	.21
4	.36	.36	.35	.35	.34	.33	.32	.30	.28	.25	.20
5	.29	.29	.29	.29	.29	.28	.28	.25	.24	.23	.19

DISPLACEMENT FACTOR FOR POISSON'S RATIO=0

Depth of point÷radius, $\frac{z}{a}$	0	0.25	0.5	0.75	1.0	1.25	1.5	2	2.5	3	4
0	2.00	1.97	1.86	1.67	1.27	0.88	0.71	0.52	0.41	0.34	0.25
0.5	1.51	1.49	1.39	1.24	1.06	.85	.68	.51	.41	.34	.25
1	1.12	1.11	1.04	.94	.85	.74	.65	.50	.40	.34	.25
1.5	.86	.85	.81	.76	.70	.65	.59	.47	.39	.34	.25
2	.68	.67	.65	.62	.59	.55	.52	.44	.38	.33	.25
3	.48	.47	.46	.45	.44	.42	.40	.37	.33	.30	.24
4	.37	.36	.35	.35	.35	.34	.33	.31	.29	.26	.22
5	.29	.29	.29	.29	.29	.28	.28	.26	.25	.24	.20

[1] Displacement, $D=\frac{\text{pressure}\times\text{radius}}{\text{modulus of elasticity}}\times$displacement factor$=\frac{pa}{E}F$; hence displacement factor, $F,=\frac{DE}{pa}$.

Table 3.—*Axial vertical stress and displacement under uniform shear stress applied over circular area*

Depth of point ÷ radius	Vertical normal stress ÷ applied stress	Displacement factor [1] (Poisson's ratio=0)
0	1.00	1.00
0.25	.91	.76
0.5	.72	.55
1	.35	.29
1.5	.17	.17
2	.09	.11
3	.03	.05
5	.01	.02

[1] Displacement $=\frac{\text{shear stress}\times\text{radius}}{\text{modulus of elasticity}}\times$factor.

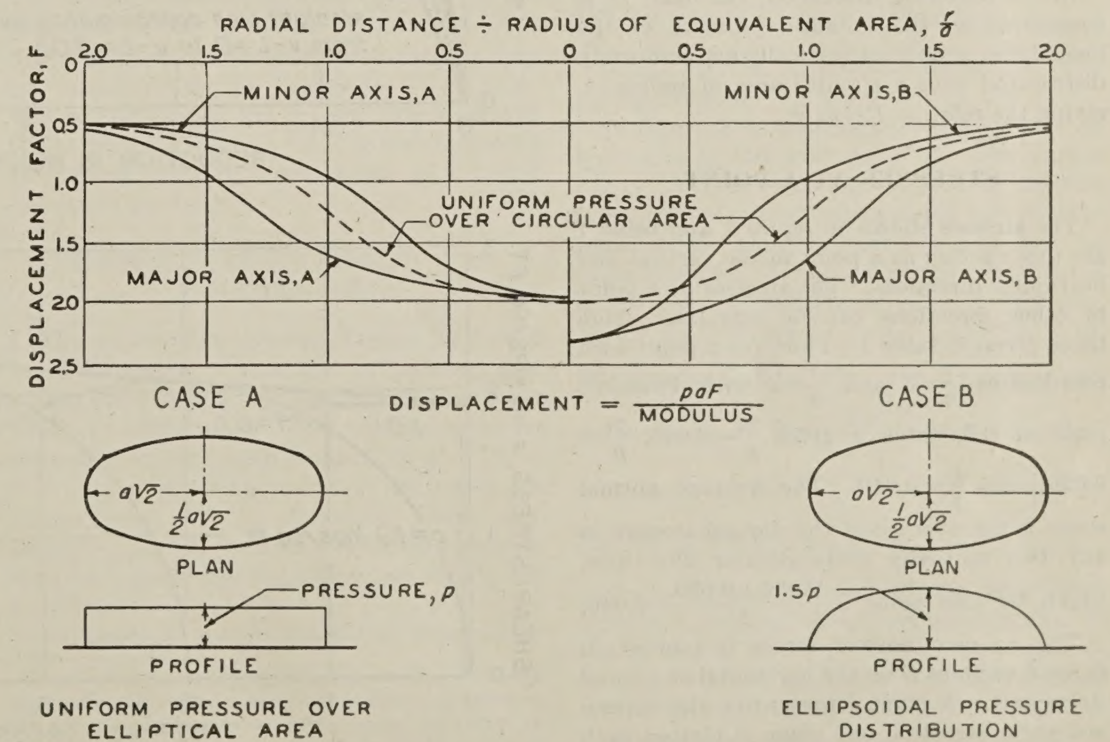


Figure 5.—Surface displacements for various pressure distributions.

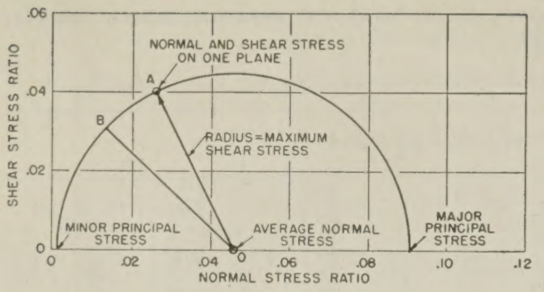


Figure 6.—Stresses at a point.

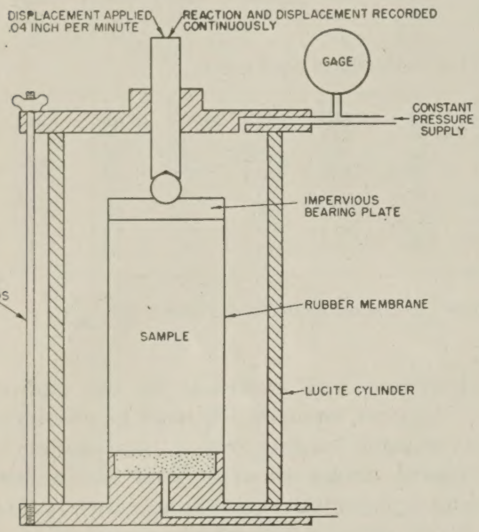


Figure 7.—Essentials of the triaxial compression test.

The theoretical surface displacements for three distributions of vertically applied pressure (*11*) are shown in figure 5. A uniform pressure over an elliptical area slightly smaller (about 10 percent) than the contact area (fig. 5, case A) is a better approximation of measured vertical pressures under pneumatic tires (*12*) than an ellipsoidal pressure distribution (fig. 5, case B). While the displacement is different on the major and minor axes, the average displacement is very close to that for a uniform pressure over a circular area.

In the following discussion, the load, P, is considered as the inflation pressure of the loaded tire, p, applied vertically and uniformly distributed over a circular area of radius, a, giving the relation $P=p\pi a^2$.

STRESSES AT A POINT

The stresses shown in figure 1 and table 1 are those acting at a point in the vertical and horizontal directions. The stresses at a point in other directions can be calculated from those given in table 1. Thus, for a point with coordinates $\frac{z}{a}=3$ and $\frac{r}{a}=2$ with Poisson's ratio of 0.5, table 1 gives $\frac{p_z}{p}=0.066$, $\frac{p_r}{p}=0.026$, and $\frac{s_r}{p}=0.040$. The average normal stress is the average of the normal stresses in any two mutually perpendicular directions, which for this point is $\frac{0.066+0.026}{2}=0.046$.

The average normal stress is plotted in figure 6 as point O on the horizontal or normal stress axis. A point representing the normal and shear stress on one plane is plotted such as point A in figure 6 for the plane perpendicular to the radial direction. Point A has an abscissa of 0.026 and an ordinate of 0.040. With point O as a center and distance OA as radius, a semicircle is drawn. Any point B on this semicircle (Mohr's circle of stress) represents the stresses acting on a plane making an angle of one-half AOB with the plane of the stresses represented by point A. The intersections of the semicircle with the normal stress axis determine the principal stresses at the point. These principal stresses act in the two mutually perpendicular directions in which there are no shear stresses. The maximum shear stress is equal to radius OA and acts in a direction at 45 degrees to the direction of the principal stresses.

In figure 6 only two dimensions have been considered, whereas there is always a third principal stress. For the case of an axially symmetrical load such as is considered here, this is the tangential normal stress and has an intermediate value.

TRIAXIAL COMPRESSION TEST

The stresses at a point in a loaded soil mass may be simulated in a cylindrical sample by applying normal stresses to its faces by means of an apparatus such as shown in figure 7 and in the cover illustration. The stress on the ends of the cylinder corresponds to the major principal stress. The stress on the curved face corresponds to the minor principal stress. The intermediate principal stress is the same as the minor principal stress, which, although an arbitrary condition, is satisfactory since it is the severest condition possible and has a minor effect. While compressive stresses are applied, the sample fails by shearing so that the test has been called both triaxial compression and triaxial shear.

To evaluate a subgrade material by means of this test, a sample may be taken from the subgrade in an undisturbed condition or prepared so as to simulate the worst conditions expected to obtain during the life of the struc-

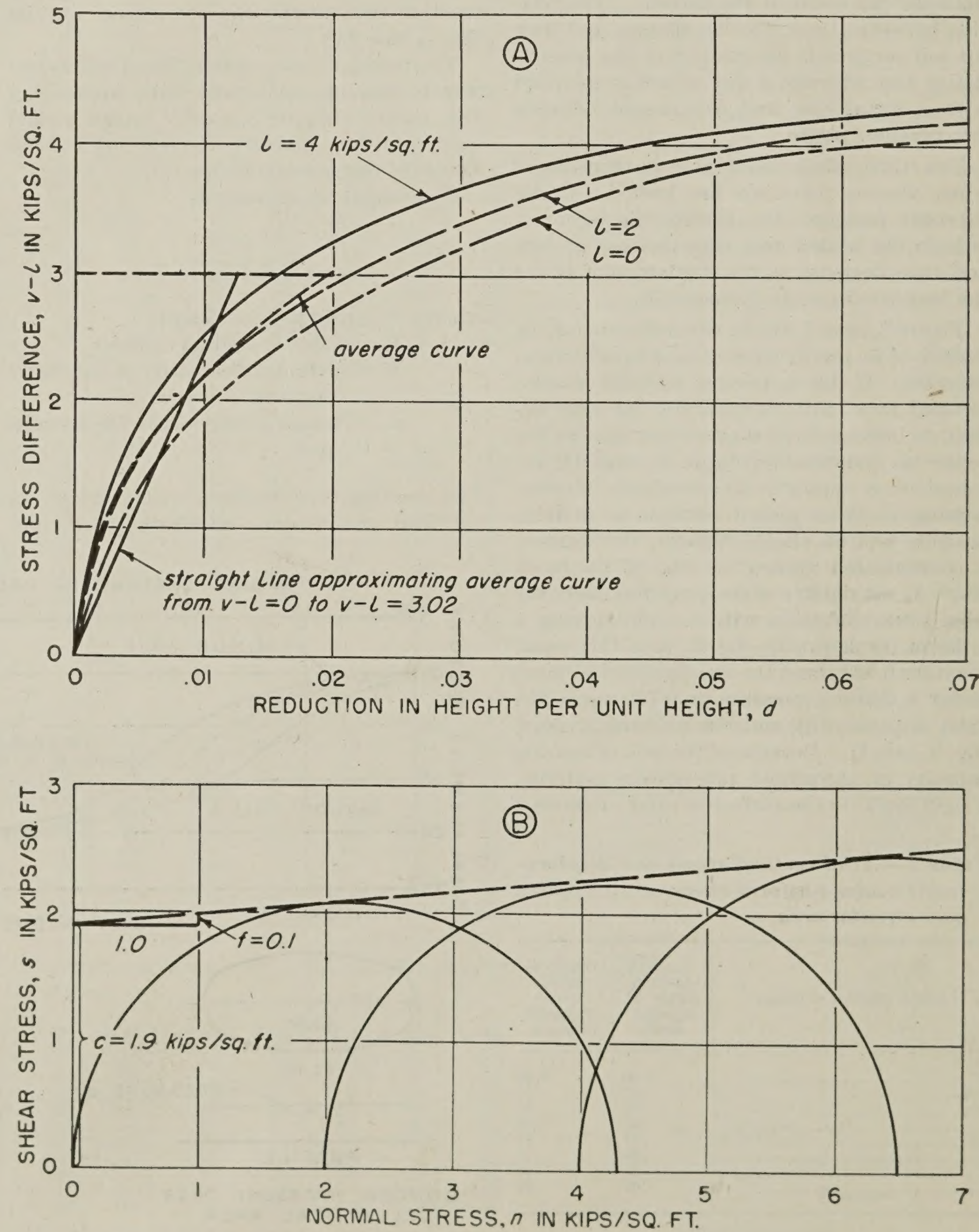


Figure 8.—Plot of triaxial compression test results.

ture and for which it is economical to design. Evaluation of field conditions is a problem in itself, involving observations at various locations of moisture, density, and temperature variations with time; and analysis of their relation to soil and pavement properties, climate, topography, ground water, and loading.

The sample is placed in a rubber membrane which is clamped to two rigid end plates, the lower of which is connected to a drain. If desired, the drain may be closed to prevent drainage from the sample. The lucite cylinder and loading head are assembled and tightened, and the head of the testing machine is brought just in contact with the piston. A constant pressure is applied to the chamber around the sample and is designated lateral pressure, l. The reaction indicator of the testing machine is set to zero. The piston is displaced at a rate of 0.04 inch per minute and the reaction and displacement recorded. The test is continued until the reaction becomes constant or diminishes. The vertical pressure, v, minus the lateral pressure, l, is calculated from the reaction, Q, the reduction in height per unit height, d, and the initial cross-sectional area of the test sample, A, by the relation

$$v-l=\frac{Q-dQ}{A}$$

Separate samples are tested with different lateral pressures to evaluate the effect of this variable. Its effect on the maximum reaction may also be determined from a single sample for materials which are not brittle by retesting the same sample at a higher lateral pressure.

The results of tests on three samples taken from a compacted clay subgrade are shown in table 4. For the three values of l, $v-l$ is plotted against d in figure 8*A*. The reduction in height due to l for $v-l=0$ represents volume change (0.2 percent for $l=2$ and 0.3 percent for $l=4$) and is not plotted in figure 8*A*, which is intended to represent the effect of distortion or lateral displacement. The curves for lateral pressures of 0, 2, and 4 kips per square foot are not far apart and an average curve will be used to calculate a modulus of elasticity.

For sand, $v-l$ is approximately proportional to l, making the construction of one effective

Table 4.—*Report of triaxial compression tests*

REDUCTION IN HEIGHT, *d*

Vertical minus lateral pressure, $v-l$ (kips per sq. ft.)	Lateral pressure, l, in kips per sq. ft.		
	0	2	4
	Percent	*Percent*	*Percent*
0 [1]	0.0	0.2	0.3
1	.3	.4	.45
2	1.0	1.0	.9
3	2.5	2.3	1.8
4	6.0	5.0	4.5

SUPPLEMENTARY TEST DATA

Maximum $v-l$ ... kips per sq. ft.	4.3	4.5	5.0
Corresponding d ... percent	10	16	17
Initial density ... lb. per cu. ft.	124	126	126
Initial moisture content ... percent of dry soil	21.8	22.3	22.5
Initial density (dry weight) ... lb. per cu. ft.	102	103	103

[1] Lateral pressure applied.

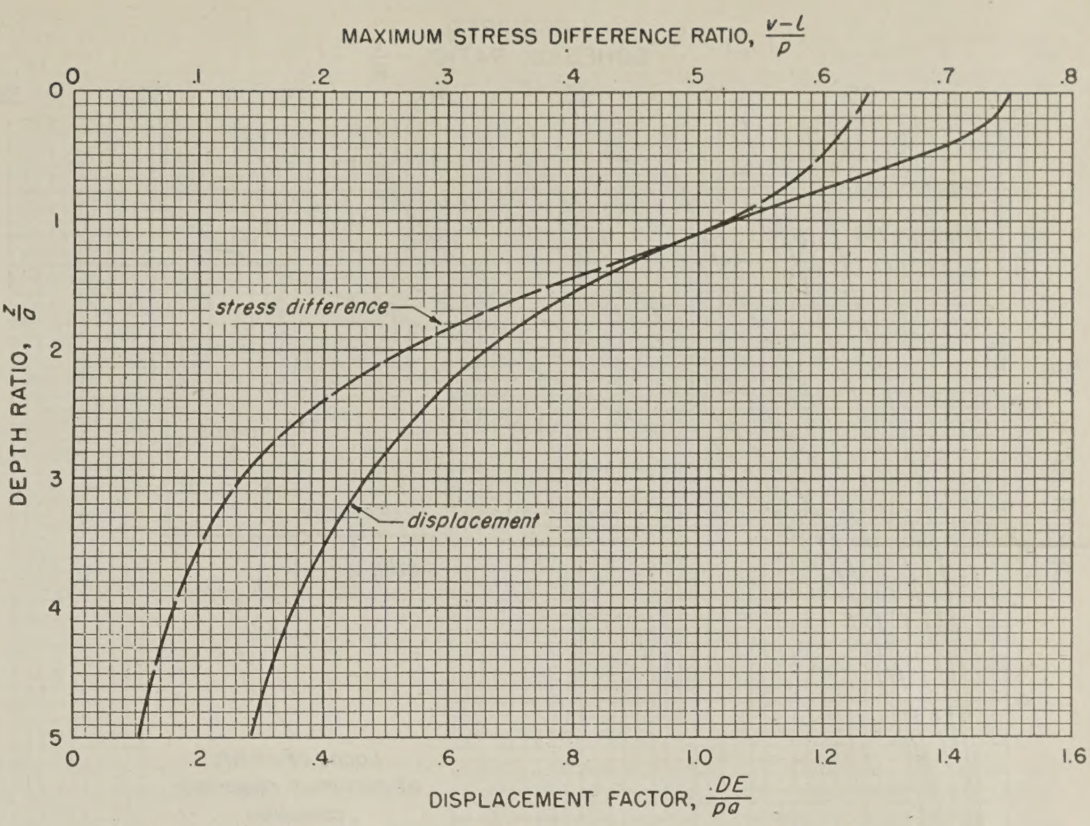


Figure 9.—Vertical displacement and stress difference relation to depth.

curve a function of l and therefore a function of the field loading to be considered.

Stress semicircles based on the principle of figure 6, using l and the maximum $v-l$ from table 4, are plotted in figure 8*B*. The minor principal stress is l, and $v-l$ equals the difference in principal stress which is the diameter of the stress circle. The circles have their centers on the horizontal axis at $l+\frac{v-l}{2}$ and pass through l plotted on the same axis. A straight line approximating the envelope of these semicircles defines a relation between the normal stresses and shear stresses on the planes of failure of the test cylinders. (The line is not tangent to all three semicircles due to variations in the samples.) The intercept of this line on the vertical axis is termed the cohesion, c, and its slope is the coefficient of internal friction, f (the angle of internal friction is arc tan f).

THICKNESS OF PAVEMENT AND SETTLEMENT [3]

The vertical displacement factors for points below the center of a uniform pressure applied over a circular area are plotted in figure 9 for Poisson's ratio=0.5. The maximum stress difference at any depth is also shown. These curves may be used to calculate the maximum displacement due to distortion below any depth for a given load on a cohesive subgrade for which triaxial compression test results are available.

For example, assume a load $P=10$ kips, with a unit pressure $p=8.64$ kips per square foot (60 pounds per square inch), and calculate the displacement at a depth of 1 foot for a subgrade represented by the test results shown in table 4 and figure 8.

[3] Settlement due to volume reduction could be calculated separately. However, since volume reduction strengthens the soil, the resulting settlement is usually not critical in pavement thickness design for distributed traffic and is not calculated here.

The equivalent radius, a, of the area over which the load is applied is calculated as follows:

$$a=\sqrt{\frac{P}{\pi p}}=\sqrt{\frac{10}{3.14\times 8.64}}=0.607 \text{ feet.}$$

At a depth of 1 foot, $\frac{z}{a}=\frac{1}{0.607}=1.65$.

From figure 9, for $\frac{z}{a}=1.65$, $\frac{v-l}{p}=0.35$ and $\frac{DE}{pa}=F$ (as defined in table 2) $=0.78$.

Then $v-l=0.35p=0.35\times 8.64=3.02$ kips per square foot.

In figure 8*A*, a straight line is drawn to approximate the average of the test curves from $v-l=0$ to $v-l=3.02$ by making the area between the vertical axis and the straight line equal to the area between this axis and the average curve. The slope of the straight line is taken as the effective modulus of elasticity, E, for the subgrade. Using the ordinate on this line at $d=0.01$ gives $E=\frac{2.35}{0.01}=235$ kips per square foot. Since $\frac{DE}{pa}=0.78$, the displacement at a depth of 1 foot is $D=\frac{0.78\ pa}{E}=\frac{0.78\times 8.64\times 0.607}{235}=0.0174$ foot or 0.21 inch.

If an allowable displacement is determined from field observations, calculations similar to the foregoing would determine the depth, z, in the subgrade at which this displacement would occur. Since E depends on the depth, z, it is generally most convenient to calculate D for various assumed values of z and plot z

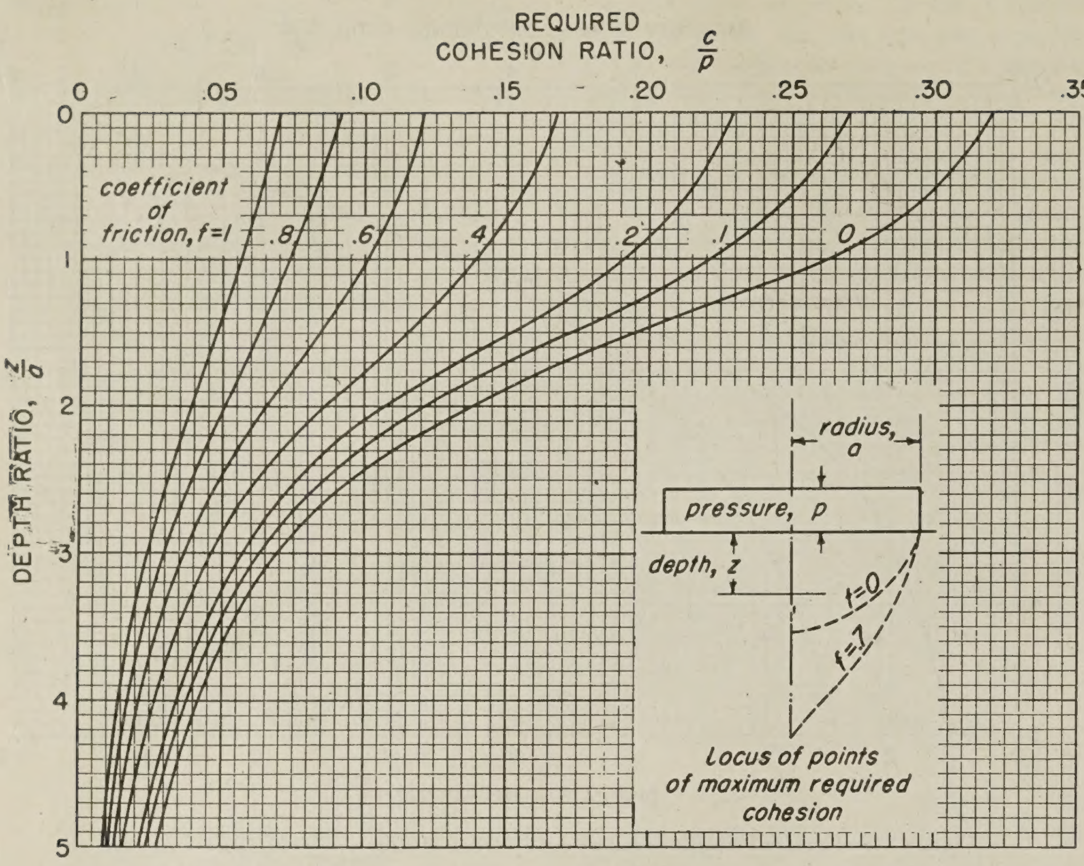


Figure 10.—Depth required to prevent overstress of a point.

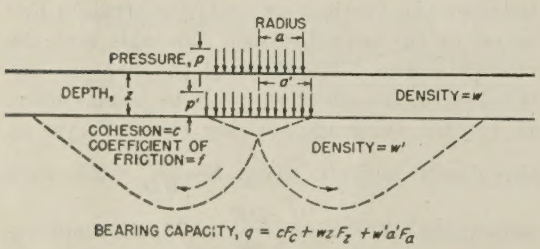
against D to determine z for given values of D. Thus, for the above conditions, at $z=1.5$ feet, $D=0.110$ inch, and at $z=2$ feet, $D=0.064$ inch. For an assumed allowable displacement of $D=0.1$ inch, $z=1.6$ feet by interpolation.

For a flexible pavement with a modulus E_p equal to that of the subgrade, this depth would be taken as the required pavement thickness (surface plus base). If E_p is greater than E, the pavement thickness, t, could be calculated from the formula (*13*):

$$t=z\sqrt[3]{\frac{E}{E_p}}$$

For several layers with different moduli, each layer can be considered as an equivalent thickness of subgrade.

For example, the Kansas Highway Department has used the above analysis (*14*) for



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	3.0
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 11.—Bearing capacity under a circular loaded area.

calculating pavement thickness using $E_p=$ 2,160 kips per square foot and $D=0.1$ inch. Using this value of E_p and $E=330$ corresponding to $z=1.6$ feet, the required pavement thickness is

$$t=1.6\sqrt[3]{\frac{330}{2160}}=0.855 \text{ feet}=10.3 \text{ inches.}$$

For granular materials the modulus varies appreciably with the lateral pressure, making it difficult to select an effective value of the modulus. For example, a base-course material conforming to the A.A.S.H.O. specifications gave a modulus of 500 kips per square foot with a lateral pressure of 1 kip per square foot, and 1,000 kips per square foot with a lateral pressure of 2 kips per square foot. Field correlations may make it possible to determine an effective lateral pressure to use in evaluating such materials.

THICKNESS OF PAVEMENT AND OVERSTRESS OF A POINT

By plotting the stress ratios from table 1 in the manner shown in figure 6, the various combinations of cohesion and friction required to prevent overstress at points at various depths were calculated (*15*) and are plotted in figure 10. Thus, for $p=8.64$ kips per square foot and $\frac{z}{a}=1.65$ as in the previous example, and with $f=0.1$ from figure 8*B*, figure 10 shows that $\frac{c}{p}=0.16$. Then the cohesion required to prevent overstress of any point in the subgrade below a depth of 1 foot is $c=0.16\times 8.64=1.38$, which is to be compared with the test value of 1.9 from figure 8*B*, giving a factor of safety of $\frac{1.9}{1.38}=1.38$.

For a factor of safety of 1, $\frac{c}{p}=\frac{1.9}{8.64}=0.22$, which from figure 10 gives $\frac{z}{a}=1.0$ or $z=1.0\times 0.607=0.607$ feet or 7.3 inches. Neglecting pavement rigidity, this is the minimum allowable thickness. Comparison of this design method with field observation has been reported from Great Britain (*16*), indicating its applicability to cohesive subgrades.

Overstress at a point occurs at a considerably lower pressure than the bearing capacity which corresponds to total failure for a single loading; however, a large number of repetitions of a load which causes overstress at a point in a clay or loose sand may eventually cause failure by progressive deterioration. In dense sands this criterion is invalid because such a material tends to expand before failure so that the stresses are redistributed from a region which is approaching overstress until the bearing capacity is reached, whereupon the sand fails suddenly in a manner typical of brittle materials.

THICKNESS OF PAVEMENT AND BEARING CAPACITY

The vertical pressure transmitted through the pavement may be compared with the bearing capacity of the subgrade (*17*) as indicatedi figure 11. The ratio of the maximum vertical pressure, p', at any depth to the pressure applied at the surface, p, is plotted in figure 12 from table 1. The pressure distribution on horizontal planes is not uniform but may be approximated for the present purpose (see fig. 11) by a uniform pressure equal to p' distributed over an area of radius a' such that the total load at any depth is equal to the load applied at the surface. The ratio $\frac{a'}{a}$ is given in figure 12.

For illustration, again take $z=1$ foot, $a=$ 0.607 foot, $\frac{z}{a}=1.65$, $p=8.64$ kips per square foot, $f=0.1$, and $c=1.9$ kips per square foot; find the density of the subgrade $w'=125$ pounds per cubic foot or 0.125 kips per cubic foot, which is the average initial density given in table 4; and assume the density of the pavement $w=0.140$ kips per cubic foot. From figure 12, for $\frac{z}{a}=1.65$, $\frac{a'}{a}=1.63$ and therefore $a'=1.63\times 0.607=1.0$ foot. Also, from figure 12, $\frac{p'}{p}=0.375$ and therefore $p'=0.375\times 8.64=$ 3.24 kips per square foot. Now the bearing capacity, q, may be computed from the formu-

Table 5.—***Effect of moisture content of clay subgrade on calculated pavement thickness***

Moisture content	Pavement thickness for 10-kip wheel load, when design criterion is—		
	Displacement	Point overstress	Bearing capacity
	Inches	*Inches*	*Inches*
14	0	0	0
19	4.2	0	4.5
23	9.8	8.6	8.0
26	12.5	15.1	13.6

la and factors given in figure 11. Thus, for $f=0.1$, $F_c=9$, $F_z=0.8$, and $F_a=0.1$, and $q=1.9\times9+0.140\times1\times0.8+0.125\times1.0\times0.1=$ 17.2 kips per square foot. Now q may be compared with p', giving a factor of safety against total failure of the subgrade of $\frac{17.2}{3.24}=$ 5.3.

For a given factor of safety, the required depth may be calculated by trial. Thus for a factor of safety of 3, the required thickness of nonrigid pavement or required effective depth of cover is $z=7$ inches.

For a cohesive material with a low value of f, as in the example above, the bearing capacity is approximately cF_c and the depth and size of loaded area have only a small effect. For sand the reverse is true. Thus, for a moderately compact sand we might have $c=0$, $f=0.8$, and $w'=0.105$ kips per cubic foot, so that with the above dimensions $q=0+0.140\times1\times67+0.105\times1.0\times51=14.8$ kips per square foot. Values of q would be higher for greater values of f which would result from increased compaction.

This method of design has been used extensively (*18*) except that the bearing capacity of the subgrade is here calculated from triaxial compression test results instead of being assumed or derived from field loading tests.

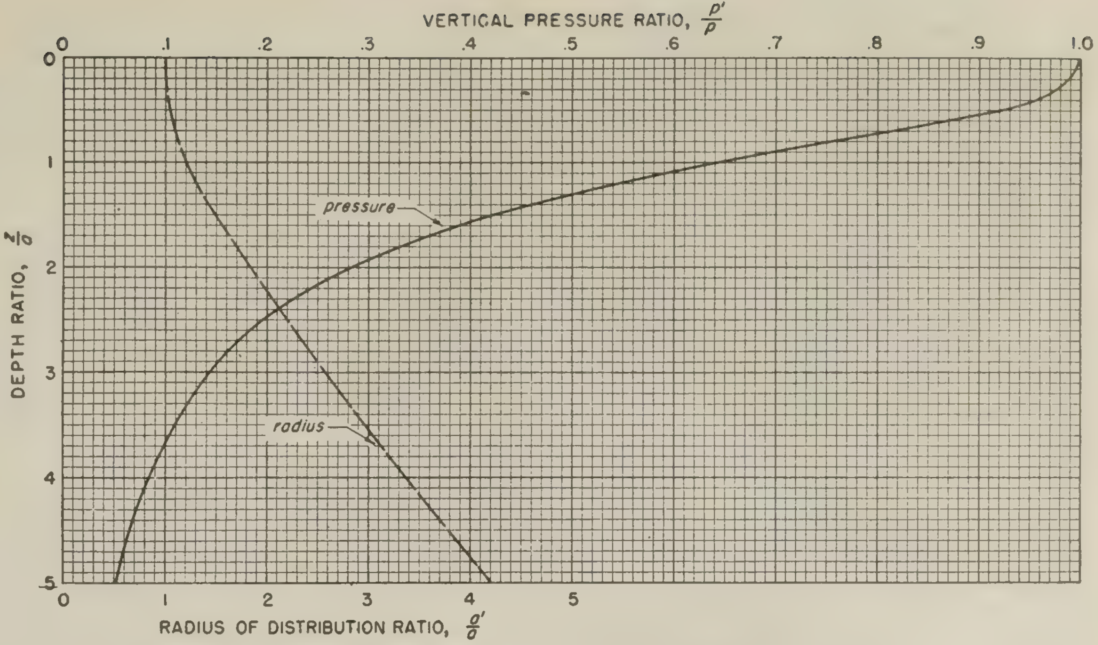


Figure 12.—Vertical pressure distribution.

EFFECT OF MOISTURE

The effect of moisture content of a single clay subgrade material on pavement thicknesses calculated by the three methods outlined above is shown in table 5. The calculated thickness depends, of course, upon the allowable settlement or required factor of safety against overstress, which in turn can only be determined by correlation of analysis of test results with pavement performance. In table 5, $P=10$ kips and $p=8.64$ kips per square foot. For the displacement method, $D=0.2$ inch and $E_p=2{,}160$ kips per square foot. For point overstress, a factor of safety of 1 is used. For bearing capacity, a factor of safety of 3 is assumed.

These analyses consider vertically applied pressures only. Therefore, the zero pavement thicknesses calculated for a moisture content of 14 percent do not indicate that no cover is required to take care of abrasive stresses. Furthermore, if such a soil were left unloaded in a humid climate, it would become much wetter and weaker. In fact, the pavement may have more effect on the strength of the subgrade than it does on the stresses transmitted to the subgrade. The moisture content of 26 percent was obtained by compacting samples to the A. A. S. H. O. maximum density and optimum moisture content and then allowing them to sorb water while confined by an all-around pressure equivalent to a head of 1 foot of water.

Table 6.—*Effect of type of subgrade on variation of pavement thickness with wheel load based on bearing capacity*

Wheel load (kips)	Pavement thickness on—	
	Clay subgrade	Sand subgrade
	Inches	*Inches*
5	9.5	8.3
10	13.6	10.0
15	16.6	10.9
20	19.2	11.5
30	23.6	12.3
60	33.3	13.3

EFFECT OF LOAD

For a constant unit pressure, the variation of pavement thickness with load calculated by the displacement method depends upon the variation of the allowable settlement with the wheel load. For instance, if the allowable D is assumed proportional to a, then t is proportional to $\sqrt{P}$. For D independent of the load, t increases much more rapidly with increased load.

For a constant factor of safety in the point overstress method, t is proportional to $\sqrt{P}$ if a constant strength of soil is assumed. As the pavement thickness increases, however, it may be expected that the minimum seasonal strength of the subgrade will increase so that t would actually increase less rapidly than $\sqrt{P}$.

Using a factor of safety of 3 against bearing capacity for the clay at 26 percent moisture with $c=1.1$ and $f=0$, and for sand with $c=0$ and $f=0.8$, pavement thicknesses for various wheel loads and $p=8.64$ kips per square foot are given in table 6. It may be seen that for the clay t is proportional to $\sqrt{P}$, but for the sand the rate of increase is much smaller.

BRAKING STRESSES

In addition to vertical pressures, the stopping of vehicles causes shear stresses, s, on the surface in the direction of travel. The magnitude of s depends upon the rate of deceleration and is limited by the product of the vertical pressure and the coefficient of friction between the tire and the surface.

The stresses transmitted from such a surface shear stress may be of use in determining the required thickness and strength of the surface course. For instance, the horizontal shear stress, s_h, near the contact of a surface course with a base course may be a factor in the required thickness of surfacing or the required strength of the materials near the plane of contact. For a uniform shear stress, s, applied over a circular area at the surface as shown in figure 13, the maximum values of s_h are in the vertical plane through the center of the circle and oriented in the direction of travel.

It so happens that the ratio $\frac{s_h}{s}$ is the same numerically as $\frac{p_r}{p}$ for Poisson's ratio=0.5 for a vertically applied load (*9*) as tabulated in table 1. Thus, for $\frac{z}{a}=0.5$, the maximum value of $\frac{s_h}{s}$ is 0.374 at $\frac{r}{a}=0$, so that the maximum shear stress in a horizontal direction at the depth $z=0.5a$ has been reduced to 37 percent of its value at the surface.

The normal stress on a horizontal plane, p_v, is compressive in front of the area to which shear stresses are applied but tensile behind the center of the loaded area. The ratio $\frac{p_v}{s}$ is equivalent numerically to $\frac{s_r}{p}$ given in table

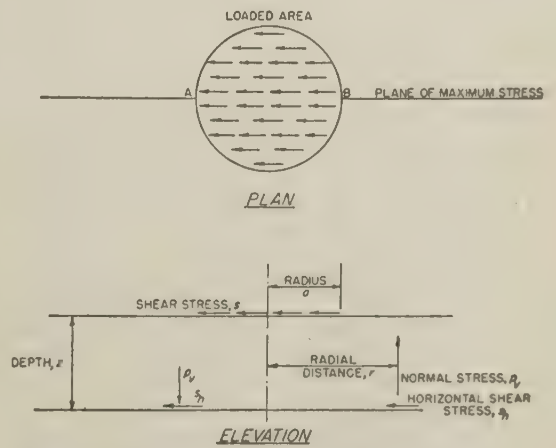


Figure 13.—Stresses from applied shear stress.

1. The maximum tensile stress is at the edge of the loaded area (point B in fig. 13) where $\frac{p_s}{s}=0.318$. At a depth $\frac{z}{a}=0.5$, the maximum value of $\frac{p_v}{s}=0.262$, as shown in table 1 at $\frac{r}{a}=1.0$. The horizontal normal stress developed by an applied shear stress is likewise compressive in front of the loaded area and tensile behind it. Its magnitude is theoretically infinite at points A and B.

The stresses transmitted from a combination of vertical and shear loads may be calculated as the sum of corresponding stresses calculated for each load separately.

The maximum resistance of a surface to horizontal displacement is the sum of its horizontal bearing capacity, Q_h, and the total shearing resistance, S_q, at its lower boundary. For the shear loading considered above, Q_h is approximately $4\ acz+1.4\ cz^2$, where c is half the compressive strength of a cohesive surface; and $S_q=s_q\pi a^2$ where s_q is the unit shearing resistance at the lower boundary. Equating Q_h+S_q to the applied shear force, $s\pi a^2$, and solving for z gives approximately $z=\frac{2}{3}\ a\ \frac{s-s_q}{c}$ as the thickness of surface required to prevent failure by horizontal displacement.

The value of s cannot exceed c because skidding is imminent for $s=c$. For the worst condition, $s_q=0$ and $s=c$, so that $z=\frac{2}{3}\ a$. Then, for a 10-kip load with $a=0.607$ foot, $z=\frac{2}{3}\times 0.607=0.4$ foot or 5 inches. With c greater than s or with an appreciable value of s_q, the required thickness would be reduced. Thus, for the above loading and $c=2s$, $z=2.5$ inches for $s_q=0$ or $z=0.5$ inch for $s_q=0.8s$. If the surface is laminated, the same analysis could be applied to the upper layer of the surface.

NOTATION

a — radius of loaded area
a' — radius of area such that total load at any depth equals load applied at surface
A — initial cross-section area
c — cohesion
d — reduction in height per unit height
D — displacement
E — modulus of elasticity of subgrade
E_p — modulus of elasticity of pavement
f — coefficient of internal friction
F — displacement factor
F_a, F_c, F_s — bearing capacity factors
l — lateral pressure
n — normal stress
p — pressure on loaded area
p' — maximum vertical pressure
p_r — radial normal stress
p_t — tangential normal stress
p_v — vertical normal stress from applied shear stress
p_s — vertical normal stress from applied vertical pressure
P — total load
P' — load per unit length
q — bearing capacity per unit area
Q — reaction
Q_h — horizontal bearing capacity
r — horizontal radial distance from load axis to element
R — radius of rigid solid cylinder
s — shear stress
s_q — unit shearing resistance
s_h — horizontal shear stress
s_r — radial shear stress
s_z — vertical shear stress
S_q — total shearing resistance
t — pavement thickness
v — vertical pressure
w — density of pavement
w' — density of subgrade
z — depth to element from loaded surface
μ — Poisson's ratio: ratio of lateral to axial strain in a simple compression test

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Scarifying hardened soil-cement base course for reconstruction

Experimental Soil-Cement Base Course in South Carolina

*BY THE DIVISION OF PHYSICAL RESEARCH
PUBLIC ROADS ADMINISTRATION*

Reported by E. A. WILLIS, Senior Highway Engineer

In the 8-year study of soil-cement base courses described in this article it was found that, under the conditions encountered, the durability of the base courses increased proportionally with the percentage of cement used. As expected, performance of soil-cement bases on friable soils was better than that on soils of plastic nature. The conclusion that admixtures of soil and granular material alone performed as well as or better than soil-cement mixtures of equal cost is not surprising, in view of the long history of successful use of stone-stabilized base courses.

FROM 1932 to 1935 the State of South Carolina constructed several experimental sections of highway using portland cement as an admixture with the soils existing in the roadbed to serve as base course for light bituminous surfaces. The excellent results obtained from that early construction contributed largely to the increase in use of soil-cement base courses throughout the State. They served to develop and improve construction procedures and control methods, and furnished data useful for determining cement requirements for different types of soils. As further information became available from the individual sections, it was evident that additional studies were necessary to aid in solving problems relative to base thickness, quantity of cement, and method of curing. Accordingly, the South Carolina State Highway Department in cooperation with the Public Roads Administration constructed an experimental road during the summer of 1938 to study the influence of these variables on the service behavior of the soil-cement base course.

This report describes the construction of the experimental road and presents conclusions drawn from study and observation of its performance under traffic for about 8 years. The 6-mile road crosses rolling country and carries an average traffic of about 500 vehicles per day. Subgrade soils encountered were all of silt-clay material. Twenty-two sections of soil-cement base course were constructed, with thicknesses of 4, 6, and 8 inches, and with cement contents varying from 3 to 11 percent. In addition, two sections were built without cement—in these, stone screenings and crushed stone were added to the soil. Eight of the soil-cement sections were cured with the aid of either tar or emulsified asphalt surface application.

CONCLUSIONS

The conclusions drawn from observation of the experimental road are stated below. They are necessarily confined to the conditions encountered in this investigation.

Portland cement used as an admixture was found to be effective in altering the physical properties of the soil, and this alteration was a major factor in the success of the soil-cement base. One important change effected

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Figure 1.—Soil-cement base-course construction: (A) Scarifying the roadbed; (B) pulverizing with a disk harrow; (C) shaping the subgrade; (D) spreading cement; (E) applying water; and (F) wet mixing with a disk harrow.

Figure 2.—Soil-cement base-course construction (continued): (A) Wet mixing with a blade grader; (B) compacting; (C) shaping the compacted mixture; (D) rolling with trucks; (E) final blading; and (F) the finished base course.

Table 1.—*Details of experimental sections*

Section No.	Length	Soil classification group	Group index range	Design thickness	Cement content by weight	Curing aid
	Feet			*Inches*	*Percent*	
1 [1]	1,320	A-7	9-13	6	7	None.
2 [1]	1,320	A-7	13-18	6	9	Do.
3	1,320	A-5, A-7	8-13	6	5	Do.
4	1,320	A-7	13-19	6	7	Do.
5	1,320	A-7	14	6	9	Do.
6	1,320	A-7	9-14	6	11	Do.
7	1,320	A-7	11-14	6	3	Tar.
8	1,320	A-6, A-7	4-10	6	5	Do.
9	1,320	A-6	2-8	6	7	Do.
10	1,320	A-7	8-20	6	9	Do.
11	1,320	A-7	14-19	6	(2)	None.
12	1,320	A-4, A-6, A-7	6-15	6	(3)	Do.
13	1,258	A-4, A-6	4	6	3	Emulsified asphalt.
14	1,320	A-4, A-7	2-14	6	5	Do.
15	1,320	A-4	1-7	6	7	Do.
16	1,320	A-4, A-6	4-8	6	9	Do.
17	1,320	A-7	13-20	6	3	None.
18	1,320	A-5, A-6	8-10	6	5	Do.
19	1,320	A-4, A-6	6-7	6	[4] 7	Do.
20	1,320	A-6, A-7	7-11	6	9	Do.
21	1,320	A-4, A-7	4-10	4	5	Do.
22	1,200	A-4	4-5	4	7	Do.
23	1,440	A-4, A-7	7-11	4	9	Do.
24	1,919	A-4, A-6, A-7	3-11	8	3	Do.

[1] Sections 1 and 2 were scarified, pulverized, and recompacted at optimum moisture content about 3 weeks after completion of original base-course construction.
[2] Admixture of stone screenings applied at rate of 300 pounds per square yard.
[3] Admixture of stone screenings and crusher-run stone applied at rate of 300 pounds per square yard.
[4] Rain interfered with original construction. The top 2 inches were pulverized, mixed with 6 percent additional cement, and recompacted.

by the addition of cement was a reduction in the plasticity indexes of the soils.

The addition of cement to the soil increased the durability of the base course, as observed in field performance and as measured in the laboratory by increased resistance to deterioration from wetting and drying and from freezing and thawing. Each increase in the proportion of cement to soil correspondingly increased the quality of performance of the base course.

Soil-cement base courses 6 inches thick, with cement content of 9 percent, and constructed on the more friable soils, gave satisfactory performance for the full 8 years. As might be expected, the soil-cement base courses on friable soils gave better performance than comparable bases on the more plastic soils.

A 3 percent cement content was not sufficient to produce required base-course stability, even when the base thickness was increased to 8 inches.

Soil-cement bases 4 inches thick were inadequate even with cement content as high as 9 percent.

Use of a surface application of asphalt emulsion or tar as an aid in curing the soil-cement base courses proved beneficial, the emulsion producing superior results.

Admixtures of granular material and soil, without cement, gave performance equal to or better than soil-cement mixtures, and at no greater cost. The excellent performance of the stone-stabilized base courses agrees with long experience in which good results have been achieved by this low-cost construction method.

DESCRIPTION OF PROJECT

The experimental road is located on State route 93 in Lancaster County, South Carolina. It begins at a point about 6 miles northeast of Nitrolee and extends in a northeasterly direction for a distance of 6.1 miles to the intersection with route U S 521 at Lancaster.

The road passes through an area of rolling hills containing soils of the Cecil series formed by weathering of the underlying crystaline rocks characteristic of the Piedmont Plateau. Surface soils in this region vary from a light gray or pale yellow fine-sandy loam to a reddish clay loam. Generally, the subsoils consist of red or yellowish-red or mottled yellow-and-red stiff clays. As the underlying rocks are approached, the soil changes to a soft disintegrated rock containing sufficient mica to impart a greasy feel. Quartz veins occur in some places and rock fragments are encountered throughout the entire profile.

A traffic count made in 1941 disclosed that an average of 424 vehicles per day traveled over the southern 17 sections and 800 vehicles per day over the northern end which includes sections 18 to 24. During 6 weeks of Army maneuvers in 1942 between 1,500 and 2,000 vehicles used this road daily, and at one time 500 tanks, varying in weight from 12 to 32 tons, were counted in a 5-hour period. An average of 559 vehicles per day was recorded in a later traffic count made during the week of January 2 to 9, 1942.

Details of the 24 test sections, each approximately one-fourth mile in length, are given in table 1. A soil-cement base was constructed on 22 sections by mixing cement with the natural soil encountered on the grade. Base thicknesses of 4, 6, and 8 inches were employed and the cement content was varied from 3 to 11 percent by weight. No special provision was made for curing the soil-cement base on 14 of the sections. On four sections curing was aided by a light coat of tar applied the day after the base was completed, and the four remaining soil-cement sections received a similar application of asphalt emulsion.

Sections 1 and 2, which contained 7 and 9 percent of cement, respectively, were scarified and repulverized about 3 weeks after they had been completed originally. Following pulverization, the moisture content was brought up to optimum and the base courses were recompacted. This was done to determine the effect of reworking a soil-cement base after hydration was well advanced.

A heavy rain occurred while section 19 was being compacted. The resulting excessive moisture made it impossible to compact the upper 2 inches of the base course. Therefore, on the following morning the upper 2 inches were loosened, pulverized, and mixed with additional cement in an amount equal to 6 percent by weight of dry soil.

Cement was not used in the base courses of sections 11 and 12. Instead, stone screenings and crushed stone were mixed with the soil. In section 11 a mixture of soil with stone screenings, in an amount equal to 300 pounds per square yard, was used. In section 12 the soil was mixed with 150 pounds per square yard of stone screenings following which crushed stone, applied at the rate of 150 pounds per square yard, was incorporated in the upper 2 inches of the soil-screenings mixture. No attempt was made to proportion the stone screenings, crushed stone, and soil in accordance with any particular method of design. The purpose in constructing sections 11 and 12 was to produce a 6-inch base course in which the cost of the stone added would equal the cost of the cement required for an equivalent thickness of soil-cement base course containing 9 percent cement.

The soils [1] occurring in the subgrade were utilized in the construction of the base course. All of them were silt-clay materials of which more than 35 percent passed the No. 200 sieve. They varied from a group A-4 soil with a group index of 1 to a group A-7-5 soil with a group index of 20. The classification and variation in group index of the soils found on each section are indicated in table 1. Analyses of typical soils are given in table 2.

[1] The soil classification used in this report is the one adopted by the Highway Research Board Committee on Classification of Materials for Subgrades and Granular Type Roads and published in the Proceedings of the Highway Research Board, Vol. 25, 1945, pp. 376-384. Reprints of the Classification are available from the Public Roads Administration.

Table 2.—*Results of tests on typical soils*

Sample No.	Sampled from—		Passing sieve—		Liquid limit	Plasticity index	Soil classification group	Group index	Maximum dry density	Optimum moisture
	Section	Station	No. 10	No. 200						
			Percent	*Percent*					*Lb./cu. ft.*	*Percent*
1	15	450	90	41	34	6	A-4	1	112	17
2	20	508	94	66	36	12	A-6	7	113	15
3	17	483	96	79	47	20	A-7-6	13	104	19
4	10	394	100	87	64	33	A-7-5	20	100	23

After a preliminary soil survey had been made, the limits for the experimental sections were established and the percentages of cement to be used were selected arbitrarily. This selection was based primarily on previous experience with soil-cement base construction in South Carolina. Subsequently, three or more samples of soil were obtained from each section for test purposes.

In addition to the mechanical analyses and physical test constants, which were determined in the Public Roads laboratories, compaction and durability tests were performed on these samples in the South Carolina State Highway Department laboratory. In general, durability tests were made on each soil with three different cement contents: that selected for the section, 2 percent above the selected percentage, and 2 percent below. The procedures followed were similar to those later adopted by the American Society for Testing Materials in Methods D 559–44 and D 560–44.

The results of compaction tests on typical soil samples are shown in table 2. Average values of the percentage of loss in weight of soil-cement mixtures after approximately 50 cycles of wetting and drying and of freezing and thawing are shown in table 3.

CONSTRUCTION METHODS

Base construction was started in May and completed in August 1938. A bituminous surface treatment was applied in September 1938 and a re-treatment in August 1939. State forces built the base and surfacing.

Construction of the soil-cement base consisted of scarifying and pulverizing the soils in the roadbed to the required depth and width, spreading the cement, mixing the cement and dry soil, applying and mixing water, compacting the mixtures, and shaping to the desired cross section. The various operations are illustrated in figures 1 and 2.

Shaping of the base commenced when the soil was compacted to the extent that the sheepsfoot rollers penetrated only 1½ inches below the surface. Shaping and compacting were continued until the surface was brought to the desired cross section and the feet of the rollers rode on the surface without penetration. The sheepsfoot rollers were then taken off the road and compaction was completed by rolling with pneumatic-tired trucks. The appearance of the surface after rolling is shown in figure 3*A*.

Table 3.—*Results of durability tests of soil-cement mixtures*

Subgrade group and cement content by weight (in percent)	Average loss after approximately 50 cycles of— Wetting—drying	Freezing—thawing
A–4 (group index 1–7) with cement content of:—	*Percent*	*Percent*
3	32	37
5	20	26
7	7	16
9	2	8
11	2	0
A–6 (group index 2–10) with cement content of:—		
3	41	65
5	27	26
7	16	15
9	12	18
11	1	1
A–7–6 (group index 7–18) with cement content of:—		
3	61	78
5	40	39
7	12	22
9	9	11
11	4	11
A–7–5 (group index 8–20) with cement content of:—		
3	100	100
5	46	41
7	20	30
9	5	12
11	1	10

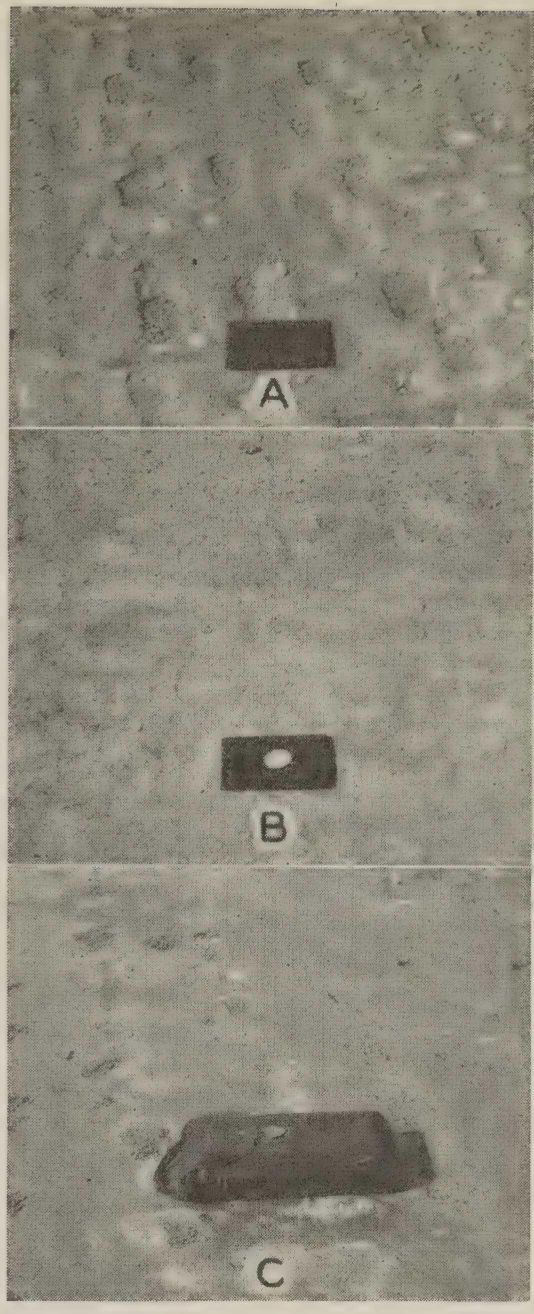


Figure 3.—Appearance of the base after rolling (A) *and after final blading on friable soil* (B) *and on heavy clay soil* (C).

Surface irregularities and roller marks were removed by blading as soon as the rolling was completed. Approximately one-half inch of material was removed in this manner. This work was done with the motor grader shown in figure 2*E* and also with a light tractor-drawn blade grader. The appearance of the surface after the final blading is illustrated in figures 3*B* and *C*: The surface texture in figure 3*B* is typical of locations where the soil was of a friable nature, while the texture in figure 3*C* is representative of the more plastic red clay soils. The scars in the surface were caused by dislodgement and removal of rock fragments during the final blading.

A transverse joint was constructed at the end of each section when the final blading started. The compacted material was first cut away to leave a smooth vertical face for the full depth of the compacted mixture. A wooden header, trimmed to conform to the final cross section of the base, was then staked against the cut face and was not removed until after the adjoining section had been constructed.

In the construction of the joint, difficulty was encountered in getting a uniformly mixed and thoroughly compacted base at the junction without damaging the completed section when the construction equipment turned around. A protective cover of earth from 2 to 3 inches thick was placed over the last 50 feet of the completed section in an attempt to overcome this difficulty. This permitted the equipment to turn on the completed section without producing any damage. However, this procedure did not eliminate the necessity for hand-mixing and tamping at the joint.

The method finally adopted at the turnarounds consisted of blading the materials forward from the joint and making all turns ahead of the completed section, as shown in figure 4. Only pneumatic-tired equipment such as water distributors and the motor patrol were permitted on the completed base course. After each mixing operation was completed, the mixture was spread out uniformly against the joint. Machine-mixing and compacting near the joint were supplemented by hand-mixing and tamping.

On sections 7, 8, 9, and 10 a surface application of tar (RT–2) was used to aid curing, while sections 13, 14, 15, and 16 were similarly treated with emulsified asphalt (South Carolina designation RCO–1). Both materials were used at the rate of about 0.2 gallon per square yard. The tar was applied by a pressure distributor on a surface previously cleaned of loose material. Emulsified asphalt was applied in the same manner as the tar, but no cleaning was necessary as the surface was maintained in a moist condition by sprinkling until the application of the emulsion. The several applications of water amounted to approximately 2.2 gallons per square yard of surface.

The remaining 16 sections were allowed to cure without a protective covering of any kind until the surface course was constructed.

Sections 1 and 2 were torn up and reconstructed about 3 weeks after they were originally constructed. The scarifier tore up the hardened soil-cement mixture in large slabs from 4 to 6 inches thick, as shown in the illustration at the top of page 9. A sheepsfoot roller reduced the slabs to small

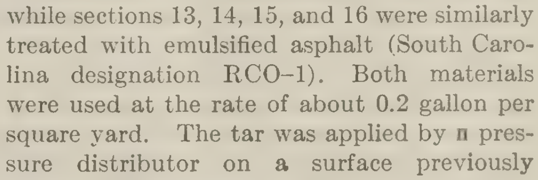


Figure 4.—Construction method at joints.

Figure 5.—**(A)** *Placing stone screenings on section 11;* **(B)** *spreading crushed stone on section 12.*

clods which were then easily pulverized by disk harrows and blade graders. As no cement was added to the pulverized soil-cement mixture, the dry-mixing operation was eliminated. Otherwise, the reconstruction was performed in the same manner as the original construction.

On section 11 the soil in the roadbed was scarified to a depth of 4 inches for a width of 21 feet. The scarified soil was then pulverized and leveled, and stone screenings were placed in piles on top of the pulverized soil as shown in figure 5*A*. The piles of screenings were spaced so that when spread uniformly over the surface an application of 300 pounds per square yard resulted. The screenings were first spread, then the soil and screenings were mixed thoroughly and uniformly, and finally the mixture was compacted and shaped to conform to the desired cross section.

A heavy rain occurred on the night previous to the mixing of the soil with the screenings. For this reason, no water was required for mixing but a great deal of extra disking and blading was necessary in order to dry out the mixture sufficiently to permit compaction. Otherwise, all the operations of scarifying, pulverizing, mixing, compacting, and shaping were performed in the same manner as described for the soil-cement mixtures.

On section 12, a 4-inch layer of the pulverized soil was mixed with stone screenings applied at the rate of 150 pounds per square yard. This mixture was shaped and compacted until the sheepsfoot rollers would penetrate only 2 inches below the surface. Crushed stone was then spread, at the rate of 150 pounds per square yard, from trucks equipped with spreader boxes as shown in figure 5*B*. The crushed stone and the 2-inch uncompacted surface layer of soil and screenings were thoroughly and uniformly mixed, shaped to cross section, and compacted by rolling with a sheepsfoot roller and pneumatic-tired trucks.

Typical gradations of the screenings and crushed stone used in sections 11 and 12 are given in table 4.

Table 4.—*Gradations of screenings and crushed stone used in sections 11 and 12*

Material passing sieve size—	Screenings	Crushed stone
	Percent	*Percent*
1½ inch		100
1 inch		96
¾ inch		67
⅜ inch		28
No. 4	100	20
No. 10	84	18
No. 40	50	9
No. 200	18	2

Observations of variations in construction procedures, weather conditions, and time required to perform the different operations on each section were made during construction of the base. Important variables have already been described. No correlation was found between weather conditions, or time of mixing, compacting, and other operations, and the condition of the base course. Therefore, since they would serve no useful purpose, weather records and time study data are not presented in this report.

BITUMINOUS SURFACE CONSTRUCTED

A bituminous wearing surface was constructed in September 1938. This consisted of a prime coat of tar at the rate of 0.22 gallon per square yard followed by an application of 0.40 gallon per square yard of hot asphalt (150–200 penetration) covered with crushed stone at the rate of 36 pounds per square yard.

In August of 1939 the experimental road was resurfaced. A tack coat of 0.10 gallon per square yard of cut-back asphalt (RC–1) was applied to the old surface and covered with 42 pounds per square yard of crushed stone. This was followed by two equal applications of sand totaling 40 pounds per square yard. When the last application of sand was dry, cut-back asphalt (RC–1) was distributed over the sand and stone at the rate of 0.90 gallon per square yard. The asphalt and aggregates were mixed by a multiple blade drag, spread, and then compacted with a 5-ton tandem roller.

CONDITION OF BASE DISCLOSED BY CORE BORINGS

At periods ranging from 7 to 40 days following construction of the various test sections, cores were drilled from the base course at 250-foot intervals. Cores were

Table 5.—*Condition of base courses after curing, as revealed by core borings*

Section Number	Designed cement content	Hardening period	Test borings	
			Number	Percent unhardened [1]
	Percent	*Days*		
1	7	30	15	0
2	9	30	18	0
3	5	19	15	60
4	7	18	15	60
5	9	16	18	50
6	11	16	15	73
7	3	40	15	7
8	5	36	15	0
9	7	35	15	0
10	9	34	18	0
11	(2)			
12	(3)			
13 [4]	3	22	15	7
14	5	21	15	0
15 [5]	7	22	18	72
16 [5]	[illegible]	19	15	40
17	3	20	12	58
18	5	17	18	6
19	7	17	15	0
20	9	16	15	7
21	5	14	18	28
22	7	24	12	0
23	9	24	18	11
24	3	7	24	13

[1] Percentage of total number of borings where base was not hard for the full processed depth.
[2] Mixture of soil and stone screenings.
[3] Mixture of soil, stone screenings, and crushed stone.
[4] Nonuniform mixing near bridge abutment.
[5] Mixed and compacted after dark.

taken on the center line and about 6 to 8 feet on either side of the center line at each location.

Many of the cores were shattered when the drill struck rock fragments in the soil. In other instances, satisfactory cores could not be obtained because the lower portions of the base-course mixtures had not hardened. For these reasons the thickness of the base courses was measured in the drill hole. The weights and volumes of the sound cores were determined later, in the laboratory, and their densities calculated.

Table 5 shows the total number of borings in each section and the percentage of the locations containing unhardened (dry, friable, and crumbly) material within the processed layer. The table also shows the age of the base at the time of drilling the cores. On all of the sections, except Nos. 3 to 7, wet-mixing during construction was accomplished by disk harrow and blade grader. On sections 3 to 7 only the disk harrow was used.

The greater uniformity of the base in the sections mixed with both disk harrow and blade grader is clearly shown in table 5. Nonuniformly hardened material was encountered at 50 percent or more of the locations drilled in sections 3, 4, 5, and 6, which had been mixed with disk harrow alone. Borings were made when these sections were 16 to 19 days old. Section 7, containing 3 percent cement, was the only other section on which mixing was accomplished solely with the disk harrow. Cores were drilled when this section was 40 days old. At that time nonhardened material was found at only 7 percent of the locations.

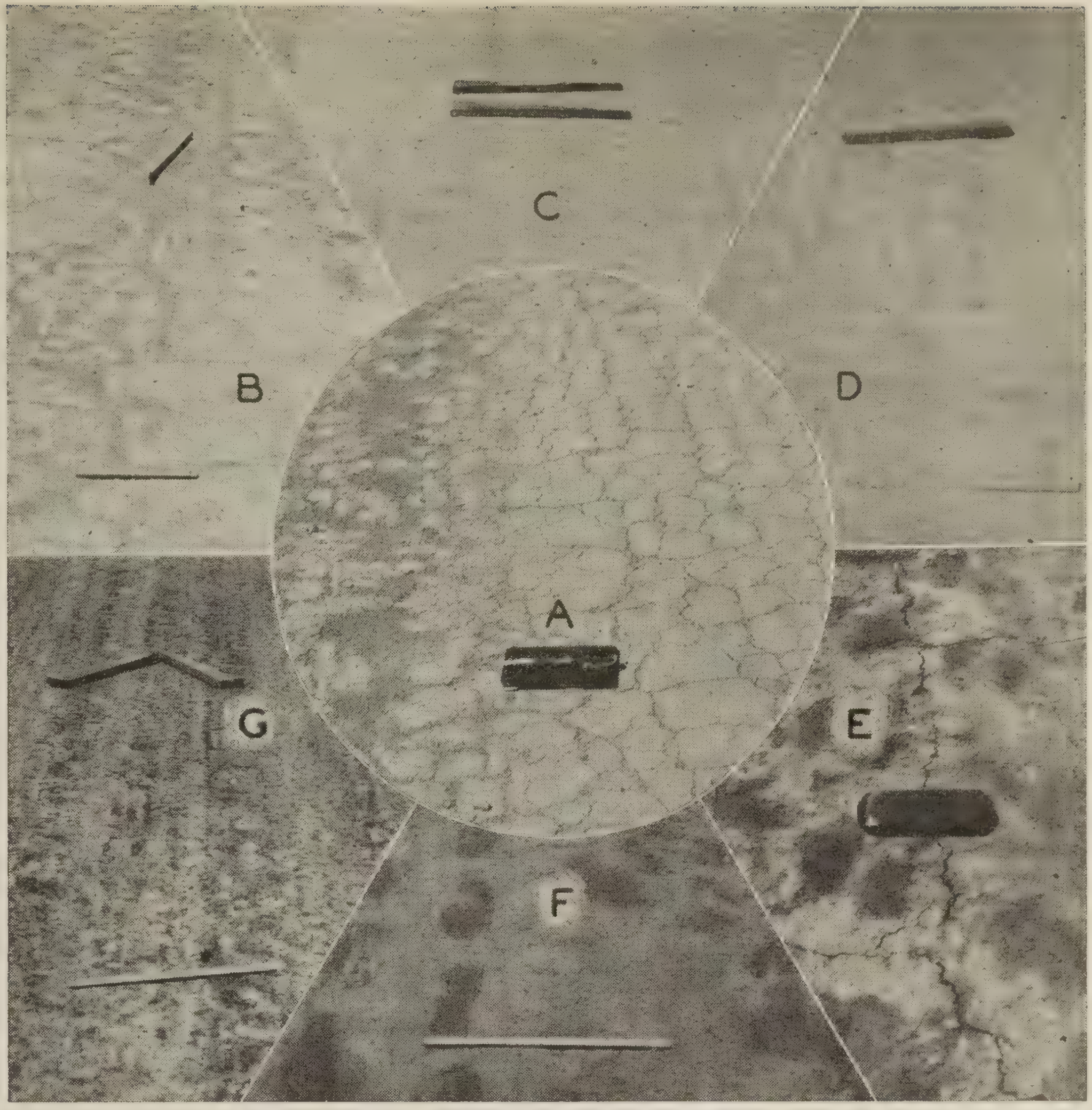


Figure 6.—**(A)** ***Shrinkage cracks in the roadbed soil prior to construction.*** **(B)** ***Appearance of the completed soil-cement base course on section 17 (3 percent cement); (C) on section 22 (7 percent cement); (D) on section 19 (7 percent cement) after the top was enriched and reconstructed; (E) on section 1 (7 percent cement) before reconstruction; (F) on the same section after reconstruction; and (G) on a section cured with emulsified asphalt.***

The disk harrow and blade grader had been used in combination in the mixing operations on the remaining 17 sections containing cement. On eight of these sections the base was uniformly hardened at all locations where borings were made. These sections were drilled at ages ranging from 17 to 36 days. Nonuniformly hardened material was encountered at some locations in all of the remaining nine sections but in only three of these, sections 15, 16, and 17, was the percentage of occurrence greater than 28 percent. Sections 15 and 16, on which nonuniform hardening was found to be 72 and 40 percent, respectively, of the core drill locations, were mixed and compacted at night with no other light than that furnished by the headlights on the construction equipment. Nonuniformly hardened base was encountered at 58 percent of the locations on section 17, containing 3 percent cement, when cores were drilled 20 days after construction.

The large number of locations where nonuniformly hardened material was encountered in the sections mixed with the disk harrow alone was due apparently to the fact that the disks did not cut deeply enough to distribute the water properly in the lower part of the mixture. This condition was corrected by using the blade grader which produced a more uniform distribution of the water in the mix.

Results of thickness and density measurements are summarized in table 6. A considerable variation from the specified thickness was found in individual cases, ranging from a minimum of 4 to a maximum of 9 inches for the 6-inch sections. However, the

Table 6.—***Summary of thickness and density measurements***

Section No.	Designed cement content	Thickness in inches				Density in pounds per cubic foot			Average compaction [1]
		Designed	Measured						
			Avg.	Max.	Min.	Avg.	Max.	Min.	
	Percent								*Percent*
1	7	6	7.0	8.0	5.0	97	103	91	92
2	9	6	7.1	9.0	5.5	97	106	92	97
3	5	6	6.7	8.5	5.2	100	108	92	100
4	7	6	7.0	8.5	5.5	100	107	95	96
5	9	6	6.9	9.0	5.2	102	107	94	101
6	11	6	7.3	9.0	4.3	98	102	91	93
7	3	6	6.5	8.0	5.0	105	109	97	101
8	5	6	6.2	7.0	4.0	100	105	90	96
9	7	6	7.0	8.5	6.0	100	107	96	92
10	9	6	6.4	9.0	5.0	92	97	90	94
11	(2)	6	6.0	7.2	5.2	119	123	117	111
12	(3)	6	6.2	7.0	5.5	122	128	116	103
13	3	6	6.0	9.0	4.0	109	116	105	97
14	5	6	6.0	8.0	4.5	108	111	105	100
15	7	6	6.4	8.5	4.5	98	100	95	94
16	9	6	6.6	8.0	5.5	101	104	96	94
17	3	6	7.3	8.0	6.5	100	100	99	100
18	5	6	6.5	8.0	5.0	97	98	95	93
19	7	6	7.2	9.0	6.0	103	106	99	94
20	9	6	6.1	8.0	5.0	101	104	100	96
21	5	4	5.2	6.5	4.5	100	104	94	93
22	7	4	5.2	6.0	4.5	101	102	100	94
23	9	4	4.5	6.5	4.0	92	97	85	89
24	3	8	7.9	9.5	6.0	101	105	93	93

[1] Average compaction expressed as percentage of average maximum density determined in accordance with Method T-99-38 of the A.A.S.H.O.
[2] Mixture of soil and stone screenings.
[3] Mixture of soil, stone screenings, and crushed stone.

average thickness of any one of this group of 18 sections was never less than 6 inches and the maximum value of average thickness was 7.3 inches. Similar variations from specified thickness were encountered on the 4-inch and 8-inch sections.

Although base-course densities (table 6) varied over a wide range, the average compaction per section, with the exception of section 23, was not less than 92 percent of the average maximum density of the soil-cement mixtures as determined in the laboratory in accordance with Method T-99-38 of the American Association of State Highway Officials. The average compaction for section 23 was 89 percent.

A check on base thickness was made in 1944 by drilling additional cores from each section at locations close to those where the original cores were taken. The individual values varied appreciably from the figures given in table 6. However, the average values for each section showed no significant change from those measured in 1938.

SHRINKAGE CRACKING

Shrinkage cracks that developed in the various sections after the base had been compacted were recorded by means of photographs.

Figure 6*A* shows the roadbed soil before it was scarified and mixed with cement. Typical examples of the cracking that developed in the soil-cement base in many of the sections where no curing aid was provided are shown in figures 6*B* and *E*. Figure 6*C* shows the appearance of section 22, where practically no shrinkage cracking occurred in the soil-cement base.

In general it was observed that as the amount of cement was increased, the amount of shrinkage cracking decreased. Differences in the amount of cracking occurring on sections having the same percentage of cement were probably due to variations in the soil.

The condition of section 19 (7 percent cement), on which the upper 2 inches had been reconstructed with an additional 6 percent of cement, is shown in figure 6*D*. It will be seen that the enriched upper layer was almost entirely free from shrinkage cracks.

The appearance of section 1 after reconstruction is illustrated in figure 6*F*. Comparing figures 6*E* and *F*, it will be seen that as a result of reconstruction the amount of cracking and the crack width had been considerably reduced.

The cracking that occurred in the soil-cement bases on sections 7 to 10, inclusive, where tar was used as a curing aid, was similar in amount and intensity to the cracking observed in the bases of corresponding cement content on the sections that had been allowed to cure without a protective cover.

Shrinkage cracking did not occur in the base courses of sections 13 to 16, where emulsified asphalt was used as a curing aid. Figure 6*G* is representative of the condition of these four sections prior to surface treatment. Unlike the tar, which penetrated to a depth of one-eighth to one-fourth inch and was dry by the following morning, the asphalt emulsion formed a film over the surface and did not dry for about 3 days. Before it dried, the emulsion film could be lifted and peeled off without disturbing the base underneath. After drying, it was securely bound to the base.

Table 7.—*Effect of cement on average liquid limits, plastic limits, and plasticity indexes*

Subgrade group and cement content (in percent)	Average liquid limit	Average plastic limit	Average plasticity index
Group A-4 with cement content of:			
0	33	26	7
3	36	32	4
5	35	33	2
7	36	[1] NP	NP
9	34	NP	NP
Group A-6 with cement content of:			
0	36	22	14
3	38	31	7
5	36	32	4
7	40	36	4
9	37	36	1
Group A-7-6 with cement content of:			
0	47	27	20
3	36	29	7
5	38	31	7
7	39	37	2
9	38	36	2
11	38	NP	NP
Group A-7-5 with cement content of:			
0	53	33	20
3	40	30	10
5	38	30	8
7	40	35	5
9	40	37	3
11	37	NP	NP

[1] NP=nonplastic.

Two other factors that probably influenced the behavior of the emulsion-treated sections were the maintenance of the surface in a moist condition by sprinkling until the emulsified asphalt was applied, and the friable character of the soils which, on an average, had lower liquid limits and plasticity indexes than any other four consecutive sections on the experimental road. Sample No. 1, table 2, is typical of the soils in these sections.

No cracking developed in the crushed stone and screenings sections, Nos. 11 and 12, except in one 140-foot length where water from heavy rains had saturated the loose mixture of soil and screenings. The mixture in this portion of the road was bladed back and forth for 5 hours until it was dry enough to roll. The moisture content, even after this period of aeration, was judged to be above optimum.

EFFECT OF CEMENT ON PLASTICITY

Samples of pulverized soil were taken from each section before cement was added. These samples, representative both of the soil in the soil-cement mixture and of the subgrade soil, were tested to determine their gradations and physical properties. The cores of hardened soil-cement previously described were obtained from the completed base course at approximately the same locations as the pulverized soil samples. After their densities had been determined, the cores were pulverized and the resulting material tested for liquid limit and plasticity index.

In all but 2 of the 66 locations sampled, the plasticity indexes of the soil-cement mixtures were appreciably lower than that of the raw soil. The plasticity indexes of the mixtures and raw soils were equal in the two exceptions noted.

The effects of the various percentages of cement on the average liquid limits, plastic limits, and plasticity indexes of soils of the A-4, A-6, A-7-6, and A-7-5 subgrade groups encountered are shown in table 7.

The cement admixtures were effective in reducing the liquid limits of the raw soils having liquid limits above about 40, while generally producing an increase in those soils having liquid limits below 40. This relation was observed regardless of the percentage of cement.

In illustration, the average liquid limits of the group A-4 and group A-6 soils without cement were 33 and 36, respectively. The same soils with cement admixtures had somewhat higher average liquid limits. The addition of cement to the group A-7-6 and A-7-5 soils, which had average liquid limits of 47 and 53, respectively, resulted in average liquid limits between 36 and 40.

The average plastic limits of all four subgrade groups were increased by the addition of cement except in the case of the group A-7-5 soils with 3 and 5 percent cement, where a slight decrease resulted. In all cases, the average plasticity indexes of the soil-cement mixtures were appreciably less than the corresponding average values for untreated soil. In general, the greater the percentage of cement, the greater was the reduction in the plasticity index, although the addition of 3 percent of cement produced a greater change than any succeeding increment.

OBSERVATIONS FROM CONDITION SURVEYS

Data relative to the service behavior of the various sections following construction of the bituminous surface were obtained in six surveys made in March 1939, March 1940, November 1940, February 1942, May 1944, and April 1946. The condition at the time of the last survey is shown graphically in figure 7. Sections having the same cement content are grouped together in this figure in order to facilitate the study of the several variable factors.

The survey of March 1939, when the road was about 7 months old, disclosed no base-course defects except in section 24 (3 percent cement) where a 75-foot length of the base was destroyed as a result of soft, unstable subgrade caused by seepage. Surface failures consisted almost entirely of peeling and pitting of the thin bituminous mat and some raveling along the edges of the surfacing. Unsatisfactory drainage caused by high shoulders that became soft and deeply rutted contributed largely towards these conditions. Surface irregularities and defects were corrected by the bituminous re-treatment placed in August 1939.

The shoulders were rutted badly at the time of the March 1940 inspection. The surfacing was in good condition throughout except that failures were beginning to occur in the sections containing 3 percent cement.

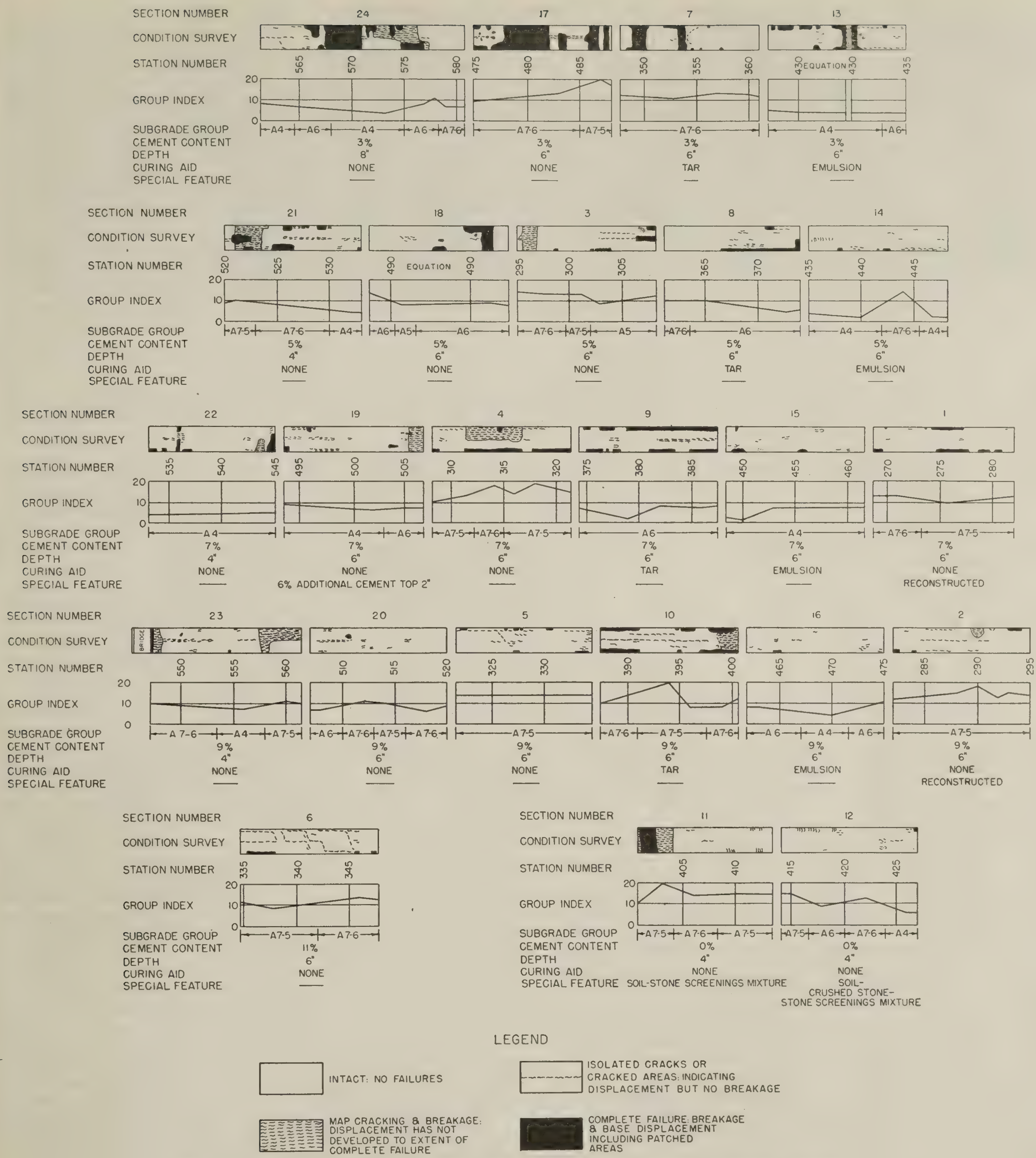


Figure 7.—April 1946 condition survey; sections grouped according to cement content.

The survey made in November 1940 revealed no significant change in the condition of the pavement.

The February 1942 survey showed that after 3½ years of service important failures had developed in the sections containing 3 percent cement. These failures consisted of breakage and displacement. Map cracking was developing in the other sections. There was also evidence that fill settlement was responsible for some of the cracking that occurred. This survey was made approximately 3 months after the completion of extensive Army maneuvers.

By May 1944, failures had increased in number and area to the extent that there was a definite indication of the difference in performance of the various sections. Additional

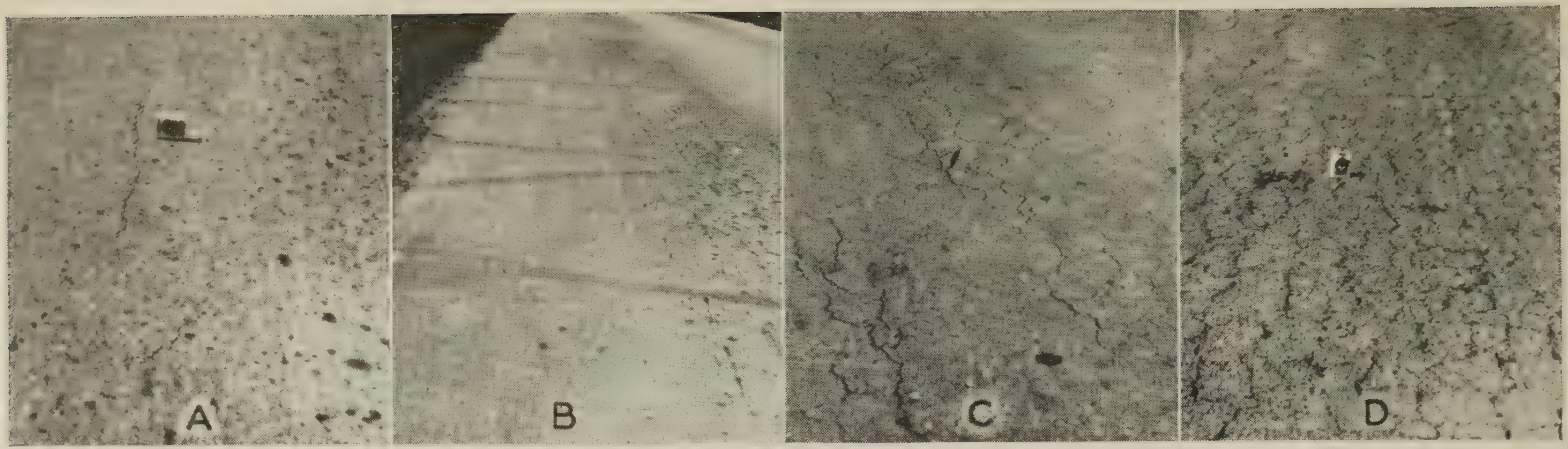


Figure 8.—Pavement performance:* (A) *Isolated longitudinal cracks;* (B) *isolated transverse cracks;* (C) *map cracking and breakage; and* (D) *complete failure.

failures occurred between the surveys of May 1944 and April 1946 but they were less in number and degree than had occurred during the previous 2 years. Most of the maintenance required between 1944 and 1946 consisted of repairing previously patched areas that continued to break up.

The varying conditions of the pavement, after approximately 8 years of service (fig. 7), were classified into four groups as follows:

Intact.—Areas containing no cracks or failures of any kind were classed as intact.

Isolated cracking.—Isolated cracks or cracked areas, indicating displacement but no breakage, are illustrated in figure 8*A* and *B*. This type of cracking was probably the result of very slight displacement produced by traffic or by volume changes in the base and subgrade. Transverse cracks, as shown in figure 8*B*, were evidently caused by shrinkage in the base or subgrade, or both. This condition did not affect the riding qualities of the road. It was important principally because of the fact that additional cracking of the same kind developed into areas of map cracking.

Map cracking and breakage.—In the classification of map cracking and breakage, a condition bordering on complete failure developed (fig. 8*C*) but was not classed as complete failure because displacement was not in evidence and riding qualities were not impaired.

Complete failure.—Complete failure due to breakage and base displacement is illustrated in figure 8*D*. This condition included all areas where the surfacing had been replaced by patches of premixed bituminous materials. The structural weakness of the base at these locations was demonstrated by the continued breaking up of the patches. Breakage was accompanied by displacement of base and surfacing and produced a rough riding surface.

Figure 7 shows general relations between the condition of the road surface and character of soil, base thickness, cement content of base, method of curing, and other factors. Many of the discrepancies in the general trends are attributable to fill settlement. While no measurements of subsidence were made, the effects of fill settlement on bridge approaches were observed soon after the road surface was constructed. Later surveys indicated that gradual settlement was taking place in all the fills, especially those over 10 feet in height, and was definitely a contributing factor in the cracking that developed.

Four sections were constructed with 3 percent cement in the base course. One of these, section 24, had an 8-inch base course, while in the remaining three, sections 7, 13, and 17, the thickness of the base was 6 inches. These four sections were continually in need of repair and, as shown in figure 7, were in the poorest condition of any group of sections on the experimental road after approximately 8 years of service. Unsatisfactory service was experienced irrespective of the type of soil from which the base was constructed, the method of curing employed, or whether the base course was 6 or 8 inches thick. Replacement of the surfacing served to improve conditions temporarily, but repeated failures in the same locations proved that the bases in these sections were structurally inadequate.

The base courses of sections 3, 8, 14, 18, and 21 contained approximately 5 percent cement. All of these sections had a designed base thickness of 6 inches except section 21, in which the nominal base thickness was 4 inches. The condition of the surface on the 5 percent soil-cement base courses after 8 years of service was better than that on the ones containing 3 percent cement. However, as shown in figure 7, a considerable portion of the area included in the 5 percent sections had failed completely, and breakage and cracking occurred in many locations. Section 21, with a 4-inch base, was the poorest of these sections. Although extensive failures occurred on fills in this section, the general distribution of failures indicates base weakness.

The full-width failure in the vicinity of station 492 in section 18 was definitely associated with fill settlement. Otherwise, this section, built on group A–5 and A–6 subgrades with group indexes generally less than 10, shows only a moderate amount of distress.

Of the remaining three 5 percent sections, section 14, built largely on group A–4 soils and cured with an emulsion cover, was in the best condition although numerous areas of isolated cracking and a few edge failures were recorded. An extensive area of breakage and cracking occurred in the vicinity of station 296 in section 3, on group A–7–6 soil. The condition of section 8, in which tar was used as a curing aid, was comparable to that of section 3, which had been allowed to cure without a protective cover.

Base courses containing approximately 7 percent cement were built on six sections, as shown in table 1 and figure 7. Extensive edge failures occurred in section 9, cured with a tar cover. Otherwise, areas of total failure were generally of smaller extent in these sections than in the sections with the same base thickness and construction variables but with 3 and 5 percent cement. Little difference could be noted in the amount of breaking and isolated cracking that occurred in the sections with 7 percent and those with 5 percent cement base courses.

The best of the 7 percent sections were section 15 (cured with an emulsion cover) and section 22, both on group A–4 soils with relatively low group indexes, and section 1 on group A–7–6 and A–7–5 soils. Section 1 was scarified to the full depth of 6 inches and recompacted about 3 weeks after the original construction. A considerable amount of cracking and breakage occurred in section 19 even though the top 2 inches of the base course were enriched with an additional 6 percent of cement following a rain during construction which interfered with proper compaction. The condition of section 4, on group A–7–5 and A–7–6 soils with high group indexes, was definitely inferior to that of other sections having bases of the same cement content but soils with lower group indexes.

Sections 2, 5, 10, 16, 20, and 23 had base courses with designed cement contents of 9 percent. The condition of these sections was generally superior to corresponding sections (those having similar base-course thicknesses, group index values, and methods of construction) having lower cement contents. Large areas of breakage and cracking in the vicinities of station 400 of section 10 and stations 547 and 560 of section 23 were associated with fill settlement.

Sections 16, 20, and 23, constructed on the more friable soils with group indexes generally less than 10, were in better condition, except in the areas of fill subsidence previously noted, than the remainder of the 9-percent sections constructed on the more plastic A–7–5 and

A–7–6 soils with group indexes generally above 10. This followed the general trend observed in the sections that have already been discussed.

Section 2, which (like section 1) was torn up and reconstructed, was superior to sections 5 and 10, which were also on soils with relatively high group indexes. The condition of the emulsion-cured section 16 was the best of any of the sections having 9 percent soil-cement bases, following another trend noted throughout the experiment.

The condition of section 6, which had a base course with a designed cement content of 11 percent, was similar to that of the adjacent section 5, with 9 percent cement and constructed on similar subgrade soils.

Fill settlement was largely responsible for the failure which occurred at the beginning of section 11. Otherwise, sections 11 and 12 were in very good condition after 8 years of service. As mentioned previously, no attempt was made to control the plasticity index or grading of the mixtures used as base courses in these sections. The principal object was to study the effect of screenings and crushed stone when used in amounts equal in cost to the cost of cement in a 6-inch base course containing 9 percent cement. Although these granular mixtures did not conform to A. A. S. H. O. specification requirements for base courses, the performance of these sections, built for the most part on group A–7–6 and A–7–5 soils having group indexes above 10, was equal to the best of the cement sections. The good performance of these sections illustrates the benefits to be derived from granular admixtures. However, it should not be inferred that the addition of crushed stone and screenings in the amounts used in this experiment will in all cases result in transforming an otherwise unsatisfactory soil into a stable base course. The construction of granular base courses without adequate control of gradation and plasticity index is not advisable and is not considered good practice.

SUMMARY

It is believed the foregoing discussion presents information that may be generally applicable to the design and construction of soil-cement bases. However, no attempt is made to draw broad conclusions since the results of one experiment of this character do not justify it. Instead, the information gathered from laboratory tests and observation of the behavior of the experimental sections of roadway is presented in summary form.

1. Portland cement was effective in changing the physical characteristics of the soils occurring on the roadway when present in the proportions used—from 3 to 11 percent.

2. In general, if the liquid limits of the raw soils were above 40 the addition of cement decreased this value, while if the liquid limits of the soils were below 40 the addition of cement resulted in a higher liquid limit (table 7). The plastic limits were with few exceptions increased by the addition of cement and in all cases a cement admixture produced a decrease in the plasticity index. Based on the average values, the addition of 11 percent cement produced a nonplastic mixture with the more plastic group A–7–6 and A–7–5 soils, while 7 percent cement was sufficient to reduce the group A–4 soils to a nonplastic condition as shown in table 7.

3. The addition of cement increased the durability of soils of the four groups encountered when subjected to either wetting and drying or freezing and thawing tests (table 3). Generally, the higher the group index, the greater the amount of cement required to produce equal durability.

4. A cement content of 3 percent did not impart to any of the soils encountered the stability required for a satisfactory base course (fig. 7). Increasing the base thickness from 6 to 8 inches, as in section 24, did not materially improve the performance of the 3 percent soil-cement mixture as a base course.

5. The performance of sections with higher cement contents in the base courses was superior to those having only 3 percent of cement. For comparable sections, i. e., those having similar base-course thicknesses, group index values, and methods of construction, there was a definite trend toward better performance with increasing cement contents (fig. 7).

6. For equal cement contents, and disregarding areas of fill subsidence, the sections constructed on the more friable soils with group indexes of 10 or less were generally in better condition after 8 years of service than those constructed on the more plastic A–7–5 and A–7–6 soils with group indexes over 10. For example, compare sections 16 and 20 with sections 2, 5, and 10, in figure 7.

7. Soil-cement base courses 4 inches thick, and having cement contents of 5, 7, or 9 percent, were not adequate on any of the soils encountered for the volume of traffic carried by this highway (fig. 7).

8. Gradual failure of soil-cement base courses 6 inches thick and with cement contents as high as 11 percent was experienced on this road on group A–7–5 and A–7–6 soils, particularly if the group index of the soil exceeded 10. For the same volume of traffic (approximately 500 vehicles per day, 15 to 20 percent of which was of the commercial type), soil-cement base courses 6 inches thick and containing 9 percent cement, appeared to be adequate on the more friable soils whose group indexes were 10 or less.

9. The use of curing aids on the soil-cement base during the hardening period appears to be desirable. In this experiment, the emulsion-cured sections were among the best of those constructed, and in every case were superior to comparable sections cured with tar. Compare sections 7, 8, 9, and 10 with sections 13, 14, 15, and 16, in figure 7.

10. The performance of sections 1 and 2 indicates that the altered physical properties of the soil produced by the addition of cement is a major contributing factor in the success of the soil-cement type base course. These sections, constructed on groups A–7–5 and A–7–6 soils with group indexes generally greater than 10, gave comparatively good results under service conditions although the hardened base course had been torn up, pulverized, and recompacted 3 weeks after it was originally constructed. Compare sections 1 and 2 with sections 4 and 5, in figure 7. After pulverizing, the cement-treated soil was very easy to reprocess with water and was similar to a friable sand-clay in workability.

11. Sections 11 and 12, whose base courses contained admixtures of granular material without the addition of cement, gave service equal to, if not better, than any of the soil-cement base-course sections.

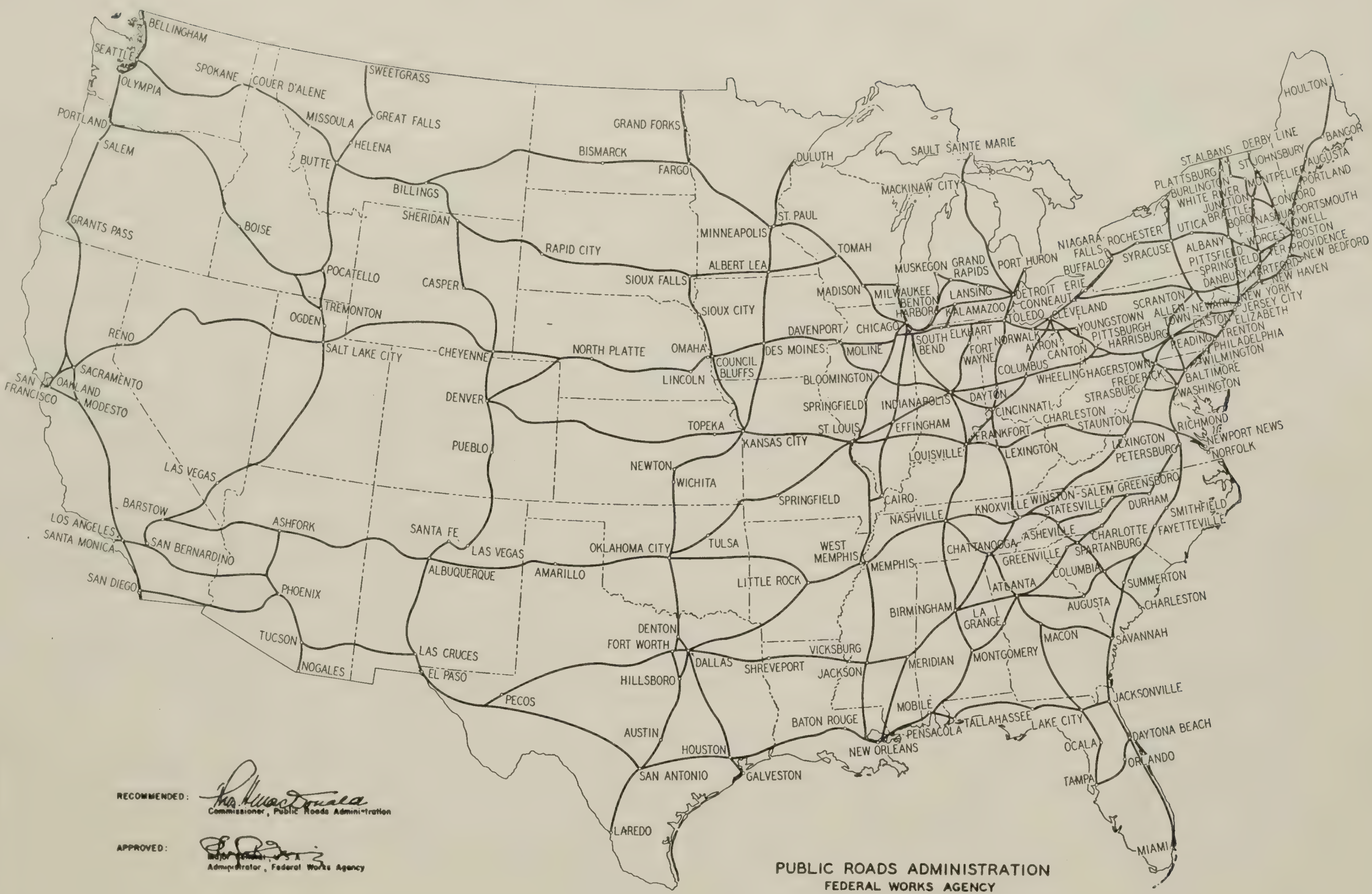
The National System of Interstate Highways

The National System of Interstate Highways (see map, p. 20), selected by the State highway departments in cooperation with the Public Roads Administration, was approved by Maj. Gen. Philip B. Fleming, Federal Works Administrator, on August 2, 1947. In recommending routes, the States were governed by the Federal-aid Highway Act of 1944 which provided for the designation of a 40,000-mile system connecting the principal centers of the country and serving the national defense.

The system as approved contains 37,681 miles of the Nation's principal highways, including 2,882 miles of urban thoroughfares. It reaches 42 State capitals and serves 182 of the 199 cities with population of 50,000 or more. All routes not already included in the Federal-aid highway system are automatically added thereto by law.

Rural sections of the interstate system comprise only 1.1 percent of all rural roads but carry 20 percent of all rural traffic. Average traffic in 1941 on the rural portions of the network was 2,693 vehicles per day, as compared with 972 on State highways.

Design standards for the system approved by the American Association of State Highway Officials call for four-lane divided highways wherever traffic is 800 vehicles or more in peak hours. Control of access, particularly in and near cities, is considered essential.



RECOMMENDED: Commissioner, Public Roads Administration

APPROVED: Administrator, Federal Works Agency

PUBLIC ROADS ADMINISTRATION
FEDERAL WORKS AGENCY

NATIONAL SYSTEM OF INTERSTATE HIGHWAYS

SELECTED BY JOINT ACTION OF THE SEVERAL STATE HIGHWAY DEPARTMENTS
AS MODIFIED AND APPROVED
BY THE ADMINISTRATOR, FEDERAL WORKS AGENCY
AUGUST 2, 1947

U. S. GOVERNMENT PRINTING OFFICE: 1947

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Act III.—Uniform Motor-Vehicle Civil Liability Act.

Act IV.—Uniform Motor-Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.