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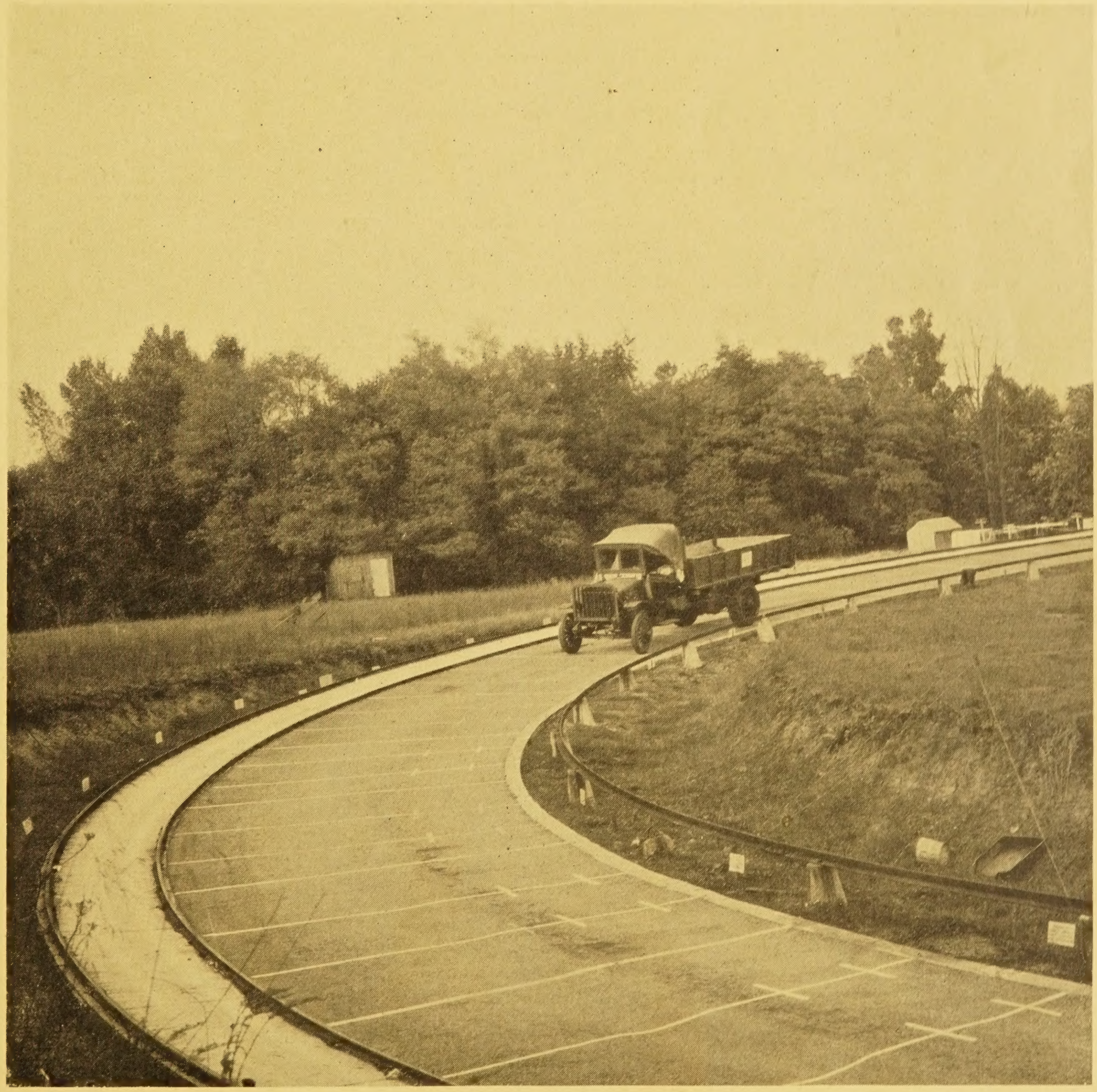

Testing Asphaltic Mixtures on Circular Track at Arlington, Va.

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STRESSES IN CONCRETE PAVEMENTS COMPUTED BY THEORETICAL ANALYSIS[1]

By H. M. WESTERGAARD, Associate Professor of Theoretical and Applied Machanics, University of Illinois

Doctor Westergaard, in the following paper prepared for presentation before the Highway Research Board, summarizes the results of a long period of study partly under the auspices of the Bureau of Public Roads. From assumptions of the conditions of loading, support, etc., of concrete road slabs, which conform closely to the actual conditions generally obtaining, he has developed by mathematical analysis a method by which the stresses in road slabs may be computed. By the use of the formulas, charts and tables which accompany the paper the method can be applied conveniently by highway engineers for the design of concrete road slabs. It also offers a means of computing the critical stresses in existing pavements, and may be used to furnish the answer to the question, often propounded, as to the possible decrease in the thickness of a pavement if the operation of the heavier vehicles is prohibited, and, vice versa, what additional thickness is required by a given increase in wheel pressure.

One may obtain a computation of stresses in concrete roads by assuming the slab to act as a homogeneous, isotropic, elastic solid in equilibrium, and by assuming the reactions of the subgrade to be vertical only and to be proportional to the deflections of the slab. With these assumptions introduced, the analysis is reduced to a problem of the mathematical theory of elasticity.

The reaction of the subgrade per unit of area at any given point may be expressed as a coefficient, k, times the deflection, z, at the point. This coefficient is a measure of the stiffness of the subgrade, and may be stated in pounds per square inch of area per inch of deflection, that is, in lb./in.^{3}. The coefficient, k, will be called the *modulus of subgrade reaction*. It corresponds to the "modulus of eleasticity of rail support" which has been used in recent investigations of stresses in railroad track.[2] The modulus, k, is assumed to be constant at each point, independent of the deflections, and to be the same at all points within the area which is under consideration.

It is true that tests of bearing pressures on soils have indicated a modulus, k, which varies considerably depending upon the area over which the pressure is distributed.[3] Yet, so long as the loads are limited to a particular type, that of wheel loads on top of the pavement, it is reasonable to assume that some constant value of the modulus, k, determined empirically, will lead to a sufficiently accurate analysis of the deflections and the stresses. One finds an argument in favor of the assumption of a constant modulus, k, for a given section of road by examining the tables which are given below. They show that an increase of k from 50 lb./in.^{3} to 200 lb./in.^{3}, that is, an increase of the stiffness of the subgrade in the ratio of four to one, causes only minor changes of the important stresses. Minor variations of k, therefore, can be of no great consequence, and an approximate single value of k should be sufficient for a quite accurate determination of the important stresses within a given section of the road. The modulus, k, enters in the formulas for the deflections of the pavements, and may be determined empirically, accordingly, for a given type of subgrade, by comparing the deflections found by tests of full-sized slabs with the defections given by the formulas.

It will be assumed for the time being that the thickness of the slab is uniform and is equal to h.

A certain quantity which is a measure of the stiffness of the slab relative to that of the subgrade occurs repeatedly in the analysis. It is of the nature of a linear dimension, like, for example, the radius of gyration. It will be called the *radius of relative stiffness*. Denoted by l, it is expressed by the formula

$$l=\sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}} \qquad (1)$$

where E is the modulus of elasticity of the concrete, and μ is Poisson's ratio of lateral expansion to longitudinal shortening. The stiffer the slab, and the less stiff the subgrade, the greater is l. One may observe that l remains constant when E and k are multiplied by the same ratio. Table 1 contains values of l for three different values of k and for different thicknesses of the slab. In computing this table as well as the three tables following, Poisson's ratio, μ, was assumed to be 0.15. This value agrees satisfactorily with the results of tests by A. N. Johnson.[4] The values of l given in the table lie between 16 and 55 inches, and about 36 inches may be considered to be a typical average.

THREE CASES OF LOADING INVESTIGATED

Figure 1 shows three cases in which it is of particular interest to be able to compute the critical stresses. In Case I a wheel load acts close to a rectangular corner of a large panel of the slab. This load tends to produce a corner break. The critical stress is a tension at the top of the slab. The resultant pressure is assumed to be on the bisector of the right angle of the corner at the small distance a from each of the two intersecting edges; the distance from the corner, accordingly is $a_1=a\sqrt{2}$. In Case II the wheel load is at a considerable distance from the edges. The pressure is assumed to be distributed uniformly over the area of a small circle with radius a. The critical tension occurs at the bottom of the slab under the center of the circle. In Case III the wheel load is at the edge, but at a considerable distance from any corner. The pressure is assumed to be distributed uniformly over the area of a small semicircle with the center at the edge and with

[1] A paper presented before the annual meeting of the Highway Research Board, National Research Council, held at Washington, D. C., Dec. 3, 1925.

[2] Progress report of the special committee to report on stresses in railroad track, Am. Soc. Civil Engineers, Trans., v. 82, 1918, p. 1191.

[3] Tests dealing with this question have been reported in a paper entitled, "Researches on the structural design of highways by the United States Bureau of Public Roads," by A. T. Goldbeck, Am. Soc. Civil Engineers, Trans., v. 88, 1925, p. 264, especially p. 271; in a paper entitled, "The supporting value of soil as influenced by the bearing area," by A. T. Goldbeck and M. J. Bussard, PUBLIC ROADS, v. 5, No. 11, Jan. 1925; and by A. Bijls, in *Génie Civil*, v. 82, 1923, p. 490. According to these tests, in the case of a pressure which is distributed uniformly over an area, the modulus, k, would be approximately inversely proportional to the square root of the area. This result is supported by theoretical considerations.

[4] "Direct measurement of Poisson's ratio for concrete," by A. N. Johnson, Am. Soc. for Testing Materials, Proc., v. 24, Part II, 1924, p. 1024.

radius a. The critical stress is a tension at the bottom under the center of the circle. In each of the three cases the load mentioned is assumed for the time being to be the only load acting.

TABLE 1.—*Values of the radius of relative stiffness, l, for different values of the slab thickness, h, and of the modulus of subgrade reaction, k, computed from equation (1)*

E=3,000,000 pounds per square inch. μ=0.15

Thickness of slab in inches h	Radius of relative stiffness, l, in inches		
	k=50 lb./in.³	k=100 lb./in.³	k=200 lb./in.³
4	23.91	20.11	16.92
5	28.28	23.78	20.00
6	32.40	27.26	22.92
7	36.40	30.60	25.73
8	40.23	33.83	28.44
9	43.94	36.95	31.07
10	47.55	40.00	33.62
11	51.08	42.94	36.11
12	54.52	45.84	38.56

For Case I a computation which may be looked upon as a first approximation was proposed by A. T. Goldbeck. Further emphasis was given to this method by Clifford Older.[5] The load is treated as a force con-

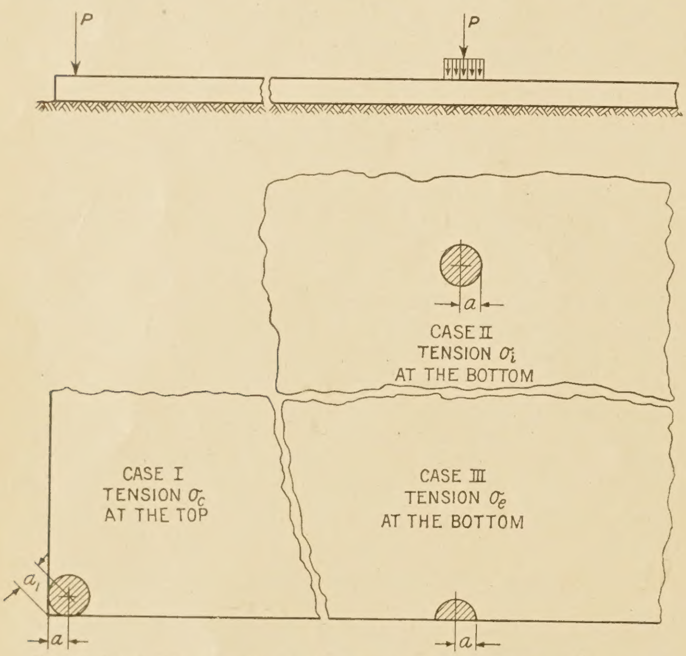


FIG. 1.—Three cases of loading. The corresponding greatest stresses are given in Tables 2 to 4

centrated at the corner itself—that is, one assumes $a=a_1=0$. At small distances from the corner the influence of the reactions of the subgrade upon the stresses will be small compared with that due to the load. The corner portion may be considered, therefore, to act as a cantilever of uniform strength. At the distance x, measured diagonally from the corner along the bisector of the right angle of the corner, the bending moment is $-Px$. This bending moment may be assumed to be distributed uniformly over the cross section, the width of which is $2x$. Thus one finds the bending moment per unit of width of cross section equal to $-\frac{P}{2}$, and the tensile stress at the top equal to

$$\sigma=\frac{3P}{h^2} \qquad (2)$$

[5] "Highway research in Illinois," by Clifford Older, Am. Soc. Civil Engineers, Trans., v. 87, 1924, p. 1180, especially p. 1206.

Since the wheel load is distributed over the area of contact between the tire and the pavement, the distances a and a_1 can not be zero. The greatest stress occurs, then, at some distance from the load. This distance will be sufficiently large to make the reactions of the subgrade outside the critical section contribute a noticeable reduction of the numerical value of the bending moment.

An improved approximation has been obtained in the following manner. The origin of the horizontal rectangular coordinates x and y is taken at the corner, the axis of x bisecting the right angle of the corner. By use of Ritz's method of successive approximation, which

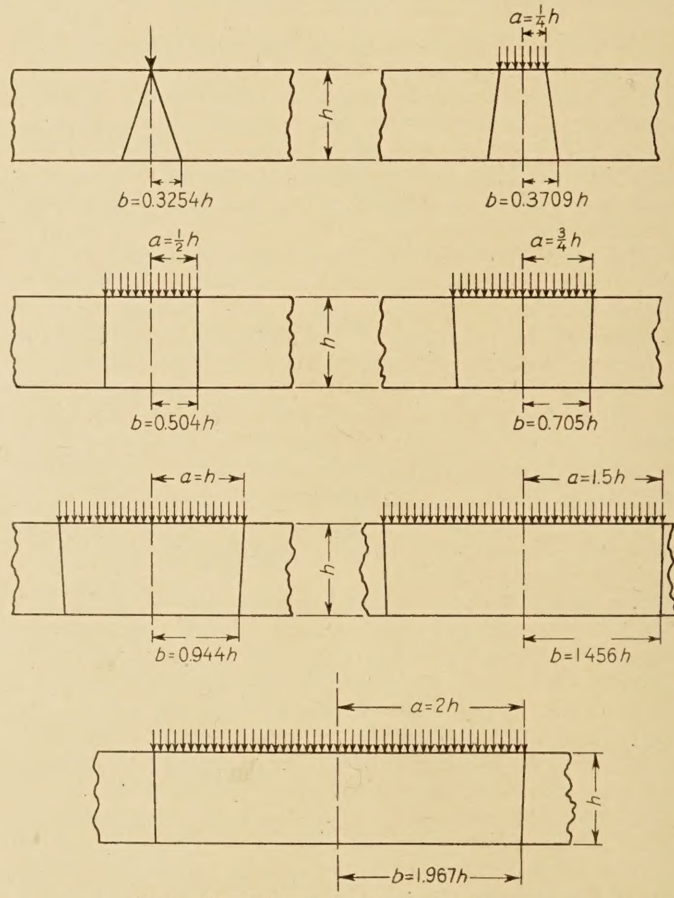


FIG. 2.—Cones of equivalent distribution of pressure

is based on the principle of minimum of energy,[6] the following approximate expression was found for the deflections in the neighborhood of the corner:

$$z=\frac{P}{kl^2}\left(1.1e^{-\frac{x}{l}}-\frac{a_1}{l}\,0.88e^{-\frac{2x}{l}}\right) \qquad (3)$$

Then the reactions of the subgrade will be expressed with sufficient exactness in terms of this function as kz. One may compute, then, the total bending moment, M', in the section $x=x_1$ due to the combined influence of the applied load and the reactions of the subgrade. When x_1 is not too large, this bending moment will be approximately uniformly distributed over the width, $2x_1$, of the cross section. That is, the bending moment per unit of width becomes $M=\frac{M'}{2x_1}$. The numerically greatest

[6] W. Ritz, Crelle's Journal, v. 135, 1909, p. 1.

value of M was found, in this manner, to occur approximately at the distance

$$x_1=2\sqrt{a_1 l} \quad (4)$$

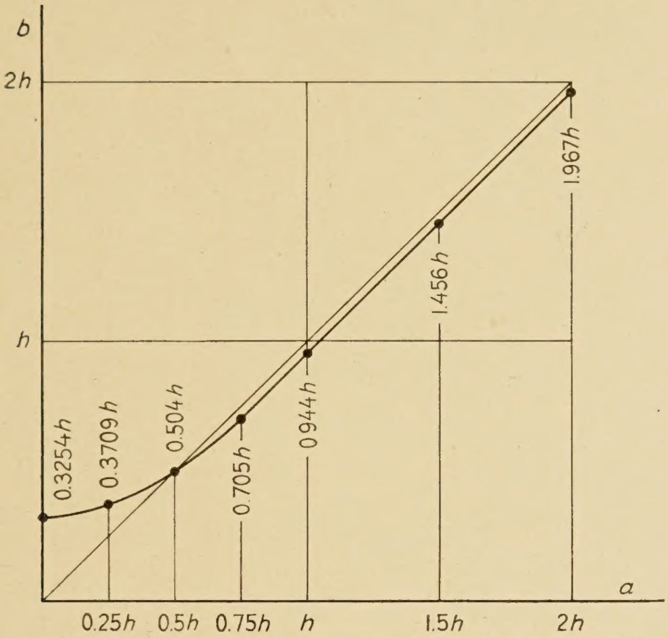


FIG. 3.—Relation between the true radius, a, the equivalent radius, b, and the thickness, h

and to be, approximately,

$$M=-\frac{P}{2}\left[1-\left(\frac{a_1}{l}\right)^{0.6}\right] \quad (5)$$

Division by the section modulus per unit of width, $\frac{h^2}{6}$, leads to the corresponding greatest tensile stress

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{a_1}{l}\right)^{0.6}\right] \quad (6)$$

This stress may be stated also in the following form which is derived by substituting the value of l from equation (1):

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{Eh^3}{12\,(1-\mu^2)\,k}\right)^{-0.15} a_1^{0.6}\right] \quad (7)$$

With $a_1=0$, the last two equations assume the simpler form of equation (2).

STRESS NOT GREATLY AFFECTED BY SUBGRADE CONDITION

Table 2 contains numerical values of the critical stress σ_c for $P=10{,}000$ pounds, $E=3{,}000{,}000$ pounds per square inch, and $\mu=0.15$. The table shows the influence of three variables: The thickness, h; the modulus of subgrade reaction, k; and the distance, a, from the edges to the center of the load.

An inspection of the table shows the influence of the variation of the distance, a, to be appreciable, amounting easily to a reduction of more than 30 per cent as compared with the value found by the first approximation, with $a=0$. The influence of the variation of the modulus, k, from 50 to 200 lb./in.^{3}, on the other hand, is not particularly large.

In Case II, that of a wheel load at a point of the interior, complications arise due to the fact that the load is concentrated within a rather small area. The theory of elasticity offers two types of theory of slabs: One that may be called the "ordinary theory of slabs," and the other, the "special theory." The difference may be explained by an analogy with beams. In analysis of beams it is assumed ordinarily that a plane cross section remains plane and perpendicular to the neutral surface during the bending. For beams of ordinary proportions, this assumption leads to satisfactory results, unless one is concerned with the local stresses in the immediate neighborhood of a concentrated load. In the latter case the assumption of the plane cross section must be abandoned, and a special theory, which takes into account the deformations due to the vertical stresses, is required. In the ordinary theory of slabs it is assumed, correspondingly,

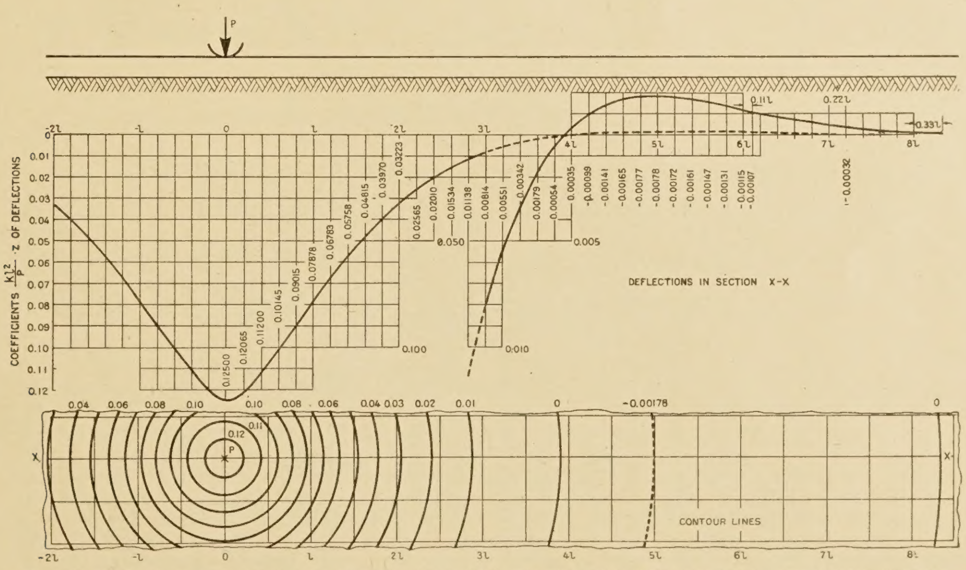


FIG. 4.—Deflections produced by a concentrated load which acts at a point of the interior at a considerable distance from the edges

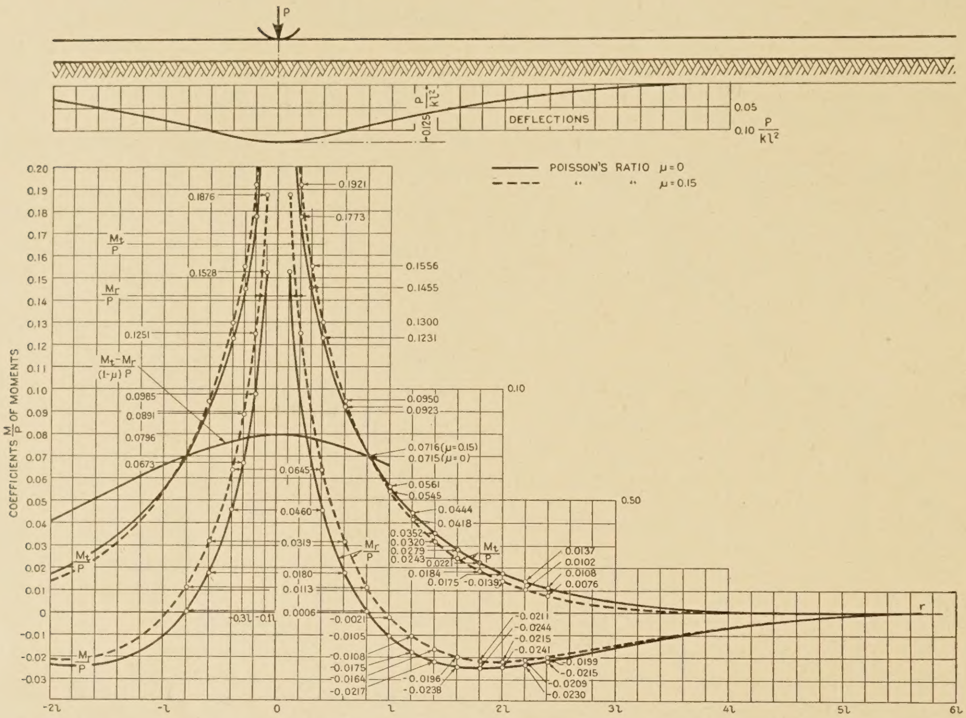


FIG. 5.—Tangential bending moments, M_t, and radial bending moments, M_r, produced by a concentrated load which acts at a point of the interior at a considerable distance from the edges

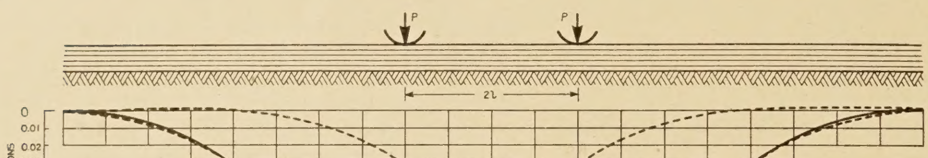


FIG. 6.—Deflections produced by two equal loads like the load in Figure 4, separated by a distance of $2l$. The deflections are found by superposition of two diagrams of the kind shown in Figure 4

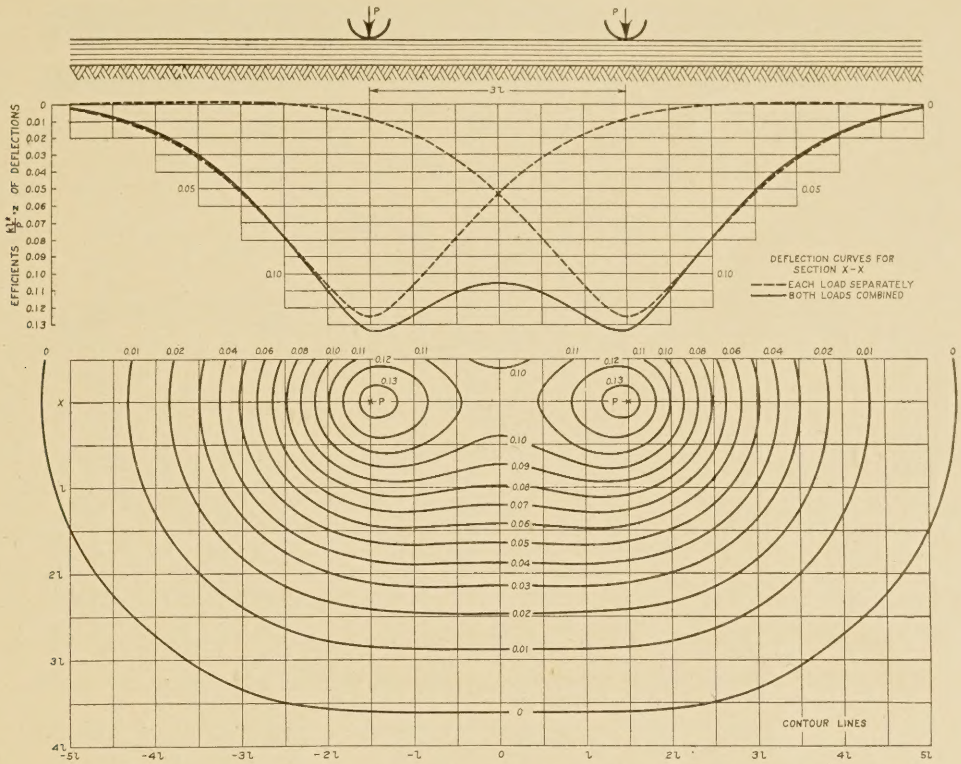


FIG. 7.—Deflections produced by two equal loads like the load in Figure 4, separated by a distance of 3l

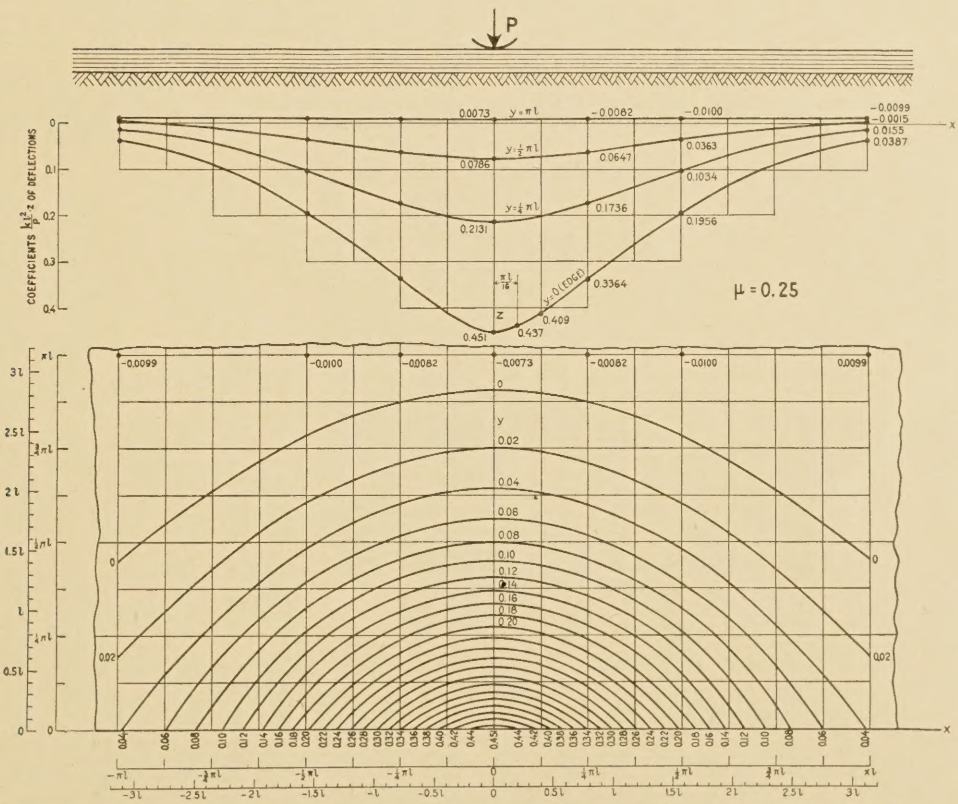


FIG. 8.—Deflections produced by a concentrated load at the edge at a considerable distance from any corner for μ=0.25

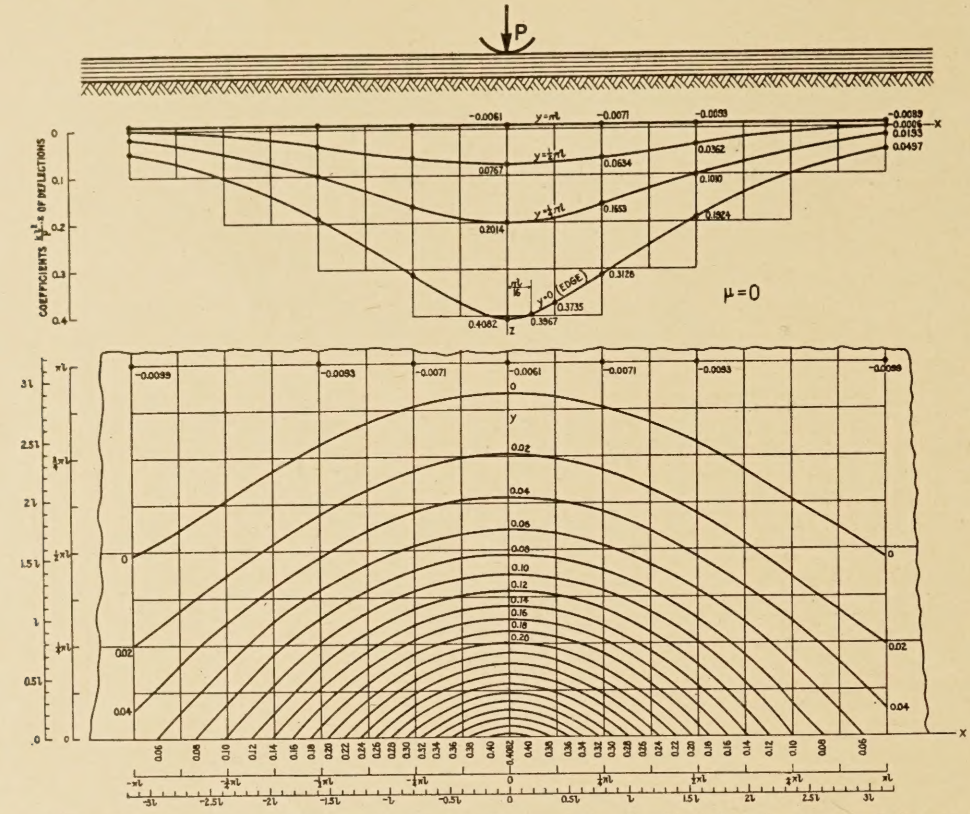


FIG. 9.—Deflections produced by a concentrated load at the edge at a considerable distance from any corner for $\mu=0$

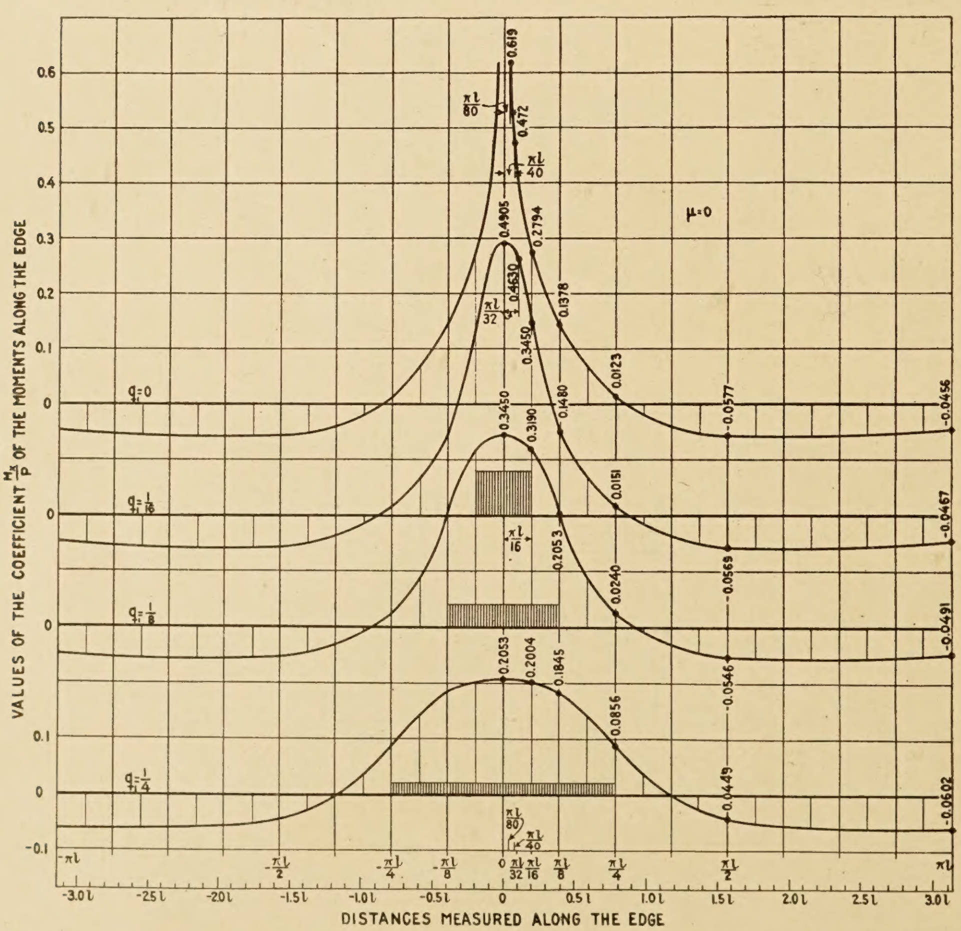


FIG. 10.—Bending moments along the edge for a load concentrated at a point of the edge (top diagram), and for loads distributed uniformly over lines of three lengths at the edge (lower three diagrams); $\mu=0$

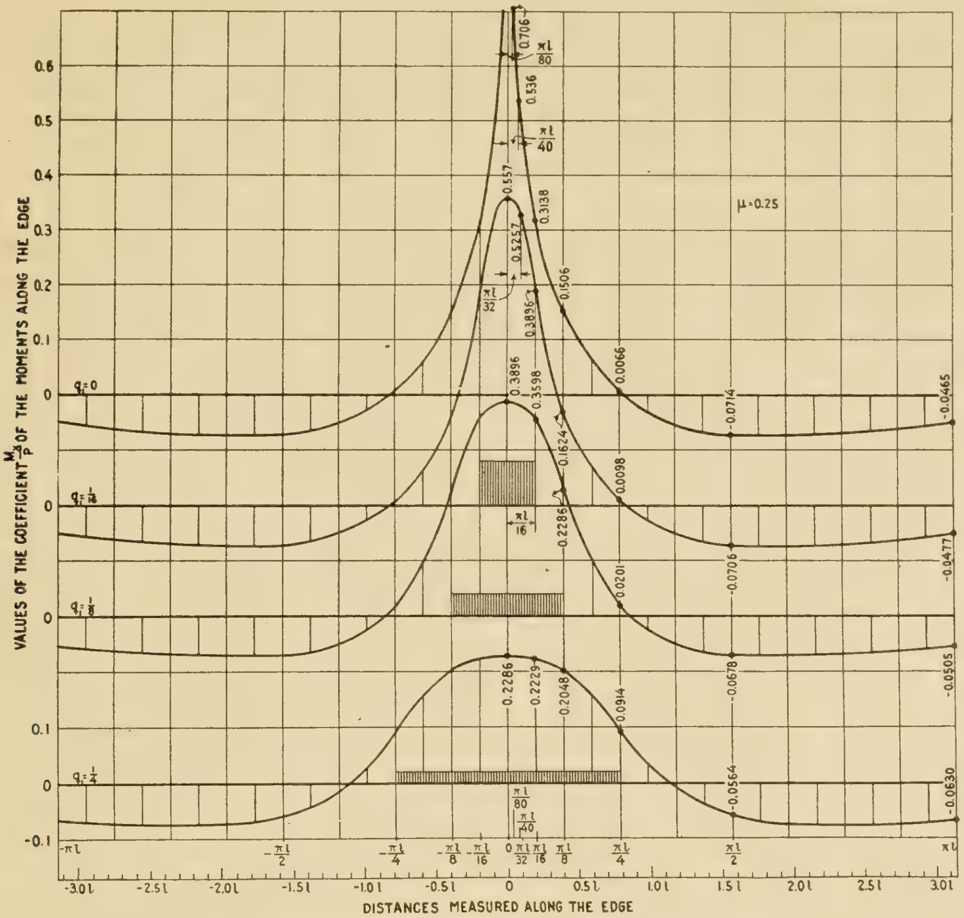


FIG. 11.—Bending moments along edge as in Figure 10, but for $\mu=0.25$

that a straight line drawn through the slab perpendicular to the slab remains straight and perpendicular to the neutral surface.

TABLE 2.—*Stresses in pounds per square inch computed from equation (7) for load condition as in Case I, Figure 1, for different values of h, k, and a*

$P=10{,}000$ pounds, $E=3{,}000{,}000$ pounds per square inch, $\mu=0.15$

Thickness of slab, h	Modulus of subgrade reaction, k	Stress in slab			
		$a=0$	$a=$ 2 inches	$a=$ 4 inches	$a=$ 6 inches
Inches	*Lb./in.*^{3}	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
6	50	833	641	541	461
	100	833	619	509	420
	200	833	596	474	375
7	50	612	480	412	357
	100	612	466	390	329
	200	612	450	366	298
8	50	469	373	325	285
	100	469	363	309	265
	200	469	352	291	242
9	50	370	299	262	233
	100	370	291	250	217
	200	370	282	237	201
10	50	300	245	216	193
	100	300	239	207	182
	200	300	232	197	169
11	50	248	204	182	164
	100	248	200	175	154
	200	248	194	167	144
12	50	208	173	155	140
	100	208	169	149	133
	200	208	165	143	124

With slabs of proportions as found in pavements, the theory based on these assumptions leads to a satisfactory determination of stresses at all points except in the immediate neighborhood of a concentrated load, and leads to a satisfactory determination of the deflections at all points. At the point of application of a concentrated force this ordinary theory leads to a peak in the diagrams of bending moments, with infinite values at the point of the load itself (as indicated in figs. 5, 10, and 11). When the force is applied at the top of the slab, the tensile stresses at the bottom are not, in fact, infinite. One may say then that the effect of the thickness of the slab is equivalent to a rounding off of the peak in the diagrams of moments. In order to find out to what extent the diagrams are rounded off, it is necessary to abandon the assumption of the straight lines drawn through the slab remaining straight, as applying to the immediate neighborhood of the load, and a special theory is required. This special theory rests on only two assumptions: One is that Hooke's law applies, the constants being the modulus of elasticity, E, and Poisson's ratio, μ; the other is that the material keeps its geometrical continuity at all points. As in the case of beams, the ordinary theory is much simpler than the special theory, and is used, therefore, except in particular cases like the present one, which deals with local effects around a concentrated load.

It is expedient to express the results of the special theory in terms of the ordinary theory in the following manner: Let the load, P, be distributed uniformly over the area of the small circle with radius a. The tensile stress produced by this load at the bottom of the slab under the center of the circle is denoted by σ_i. This stress is the critical stress except when the radius, a, is so small that some of the vertical stresses near the top become more important; the latter exception need not be considered, however, in case of a wheel load which is applied through a rubber tire. By use of the ordinary theory one may find the same stress at the same place by assuming the load to be distributed over the area of a circle with the same center, but with the radius b. One finds that this equivalent radius, b, can be expressed with satisfactory approximation in terms of the true radius, a, and the thickness, h, only.

In order to find the relation between h, a, and b, numerical computations were made in accordance with an analysis which is due to A. Nádai.[7] The center of the load P is assumed for the time being to be at the center of a circular slab. The slab is supported at the edge in such a manner that the sum of the radial and tangential bending moments is zero at every point of the edge. Computations according to Nádai's analysis, with the radius of the slab equal to $5h$, gave the results which are represented in Figure 2 in the manner of "cones of equivalent distribution" and in Figure 3 by a curve with coordinates a and b. Approximately the same cones and the same curve are obtained for other radii of the slab; and the results may be applied generally to slabs of proportions such as are found in concrete pavements, with any kind of support which is not concentrated within a small area close to the load.

One may notice that when a increases gradually from zero, b is at first larger than a; but when a passes a certain limit, b becomes smaller than a. For the larger values of a, the ratio, $\frac{b}{a}$, converges toward unity, and the ordinary theory of slabs, accordingly, gives nearly the same results as the special theory.

The curve in Figure 3 is found to lie close to a hyperbola, the equation of which may be written in the following form, which is suitable for numerical computations, and which may be used for values of a less than $1.724h$:

$$b=\sqrt{1.6a^2+h^2}-0.675h \qquad (8)$$

For larger values of a, one may use $b=a$, that is, the ordinary theory may be used without corrections.

By the ordinary theory one finds the following approximate expression for the critical stress:

$$\sigma_i=\frac{3(1+\mu)}{2\pi h^2}P\left(\log_e\frac{l}{a}+0.6159\right) \qquad (9)$$

With $E=3{,}000{,}000$ pounds per square inch and $\mu=0.15$, and with l substituted from equation (1), this formula takes the form:

$$\sigma_i=0.3162\frac{P}{h^2}[\log_{10}(h^3)-4\log_{10}a-\log_{10}k+6.478] \qquad (10)$$

[7] A. Nádai, See "Die Biegungsbeanspruchung von Platten durch Einzelkräfte," Schweizerische Bauzeitung, v. 76, 1920, p. 257; and his book, "Die elastischen Platten," (Berlin) 1925, p. 308.

The correction to be made in this formula in order to make it agree with the special theory is merely to replace the true radius, a, by the equivalent radius, b. Thus one finds the following formula, which replaces equation (10) when a is less than $1.724h$:

$$\sigma_i=0.3162\frac{P}{h^2}[\log_{10}(h^3)-4\log_{10}(\sqrt{1.6a^2+h^2}-0.675h)-\log_{10}k+6.478] \qquad (11)$$

The stresses given in Table 3 have been computed in accordance with this formula for $P=10{,}000$ pounds. Like Table 2, this table shows the influence of three variables: the thickness, h; the modulus of subgrade reaction, k, and a. In Table 3, as in Table 2, one may notice the relatively greater influence of the variation of a as compared with the influence of the variation of k.

In dealing with Case III, that of a wheel load at the edge, it was assumed that an equivalent radius, b, may be introduced in the place of the true radius, a, in the same manner as in the preceding case, and by the same formula, that of equation (8). This assumption may be justified on the ground of the similarity in the two cases in the distribution of the energy due to vertical shearing stresses. By introducing the equivalent radius, b, in the place of a in the formula for the tensile stress, σ_e, along the bottom of the edge under the center of the circle, as obtained by the ordinary theory, one finds the following expression which, like the analogous equation (11), is based on $E=3{,}000{,}000$ pounds per square inch and $\mu=0.15$:

$$\sigma_e=0.572\frac{P}{h^2}[\log_{10}(h^3)-4\log_{10}(\sqrt{1.6a^2+h^2}-0.675\,h)-\log_{10}k+5.767] \qquad (12)$$

Stresses computed according to this formula are given in Table 4, again for $P=10{,}000$ pounds. The influence of the three variables, h, k, and a, is shown in the same manner as in the two preceding tables, and is seen to be of the same nature, the variation of a being of greater importance than that of k.

TABLE 3.—*Stresses in pounds per square inch computed from equation (11) for load condition as in Case II, Figure 1, for different values of h, k, and a*

$P=10{,}000$ pounds, $E=3{,}000{,}000$ pounds per square inch, $\mu=0.15$

Thickness of slab h	Modulus of subgrade reaction k	Stress in slab				
		$a=0$	$a=2$ in.	$a=4$ in.	$a=6$ in.	$a=8$ in.
Inches	*Lb./in.*^{3}	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
4	50	1,231	1,058	848	693	588
	100	1,172	998	788	634	528
	200	1,112	939	729	574	469
5	50	763	694	580	487	415
	100	725	656	542	449	377
	200	687	617	504	411	339
6	50	523	487	421	361	313
	100	497	461	395	335	287
	200	470	435	368	308	260
7	50	380	360	319	279	245
	100	361	341	300	260	226
	200	341	321	280	240	206
8	50	288	276	250	222	197
	100	273	261	235	207	182
	200	258	246	220	192	167
9	50	226	218	200	180	162
	100	214	206	188	169	150
	200	202	194	177	157	138
10	50	181	176	164	149	136
	100	172	167	154	140	126
	200	162	157	145	130	116

TABLE 4.—*Stresses in pounds per square inch computed from equation (12), for load condition as in Case III, Figure 1, for different values of h, k, and a*

P=10,000 pounds, E=3,000,000 pounds per square inch, μ=0.15

Thickness of slab h	Modulus of subgrade reaction k	Stress in slab				
		$a=0$	$a=2$ in.	$a=4$ in.	$a=6$ in.	$a=8$ in.
Inches	*Lb./in.*³	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
6	50	833	769	649	541	453
	100	785	721	601	493	406
	200	738	673	553	445	358
7	50	604	568	494	422	360
	100	569	533	459	386	325
	200	534	498	424	351	290
8	50	457	436	388	337	293
	100	430	409	361	311	266
	200	404	382	334	284	239
9	50	358	344	312	276	243
	100	337	323	291	255	222
	200	315	301	269	233	200
10	50	287	278	256	230	204
	100	270	261	239	212	187
	200	253	244	221	195	170
11	50	235	229	213	194	174
	100	221	215	199	180	160
	200	207	201	185	165	146
12	50	196	192	180	165	150
	100	184	180	168	153	138
	200	172	168	156	142	126

BALANCED DESIGNS TESTED BY USE OF TABLES

From the three tables, for Cases I, II, and III, one may obtain suggestions on the question of balanced design. Consider, for example, a pavement with the thickness of 7 inches in the interior portion, and 9 inches at the edges. It may be assumed for the time being that the outer portions behave as a large slab with uniform thickness of 9 inches. With the thickness diminishing slowly toward the interior, the stresses σ_c and σ_e would be somewhat larger than with constant thickness of 9 inches, but the correction needed for this reason is probably only small. For the time being only the one wheel load which is considered in each of the three tables will be taken into account. The influence of other wheel loads acting on the same panel, but at some distance, will be considered later; in any case it is found to be relatively small. With $P=10,000$ pounds, $k=50$ lb./in.³, and $a=4$ inches, the three tables give the following values:

$$\sigma_c=262 \text{ lb. per sq. in.}, \sigma_i=319 \text{ lb. per sq. in.}, \sigma_e=312 \text{ lb. per sq. in.}$$

In comparing these stresses, their different characters should be considered. The stress, σ_c, at the corner acts presumably throughout the width of a whole cross section, whereas σ_i and σ_e are localized within smaller regions. With equal tendency to rupture at the three places, σ_c then, should be, probably, somewhat smaller than σ_i and σ_e. The stress, σ_e, is produced under the influence of a load which is distributed over an area only one-half of that assumed for σ_i. Although the situation represented by the smaller area may occur when a wheel moves in over the edge of the pavement, it is reasonable, for the purpose of a comparative study of the tendency to rupture, to assume a larger radius of the semi-circle at the edge than for the full circle in the interior portion. With $a=6$ inches, for example, at the edge, one finds the stress

$$\sigma_e=276 \text{ lb. per. sq. in.}$$

In comparing this stress with σ_i it should be observed that σ_i represents a state of equal stresses in all horizontal directions at the point, whereas σ_e is a one-directional stress. The elongations per unit of length are in the two cases $\frac{\sigma_i(1-\mu)}{E}$ and $\frac{\sigma_e}{E}$. It appears to be reasonable, therefore, for the purpose of comparison, to replace σ_i by an equivalent one-directional stress; if in this case the elongation is a direct measure of the tendency to rupture, this equivalent stress should be

$$\sigma_i'=\sigma_i\ (1-\mu)=319\ (1-0.15)=271 \text{ lb. per sq. in.}$$

The three values 262, 271, and 276 pounds per square inch point toward the conclusion that the assumed design is suitably balanced.

The suggestion has been made already that one may determine suitable values of k by comparing the deflections found by tests of full-sized slabs with those given by the formulas. The following formulas lend themselves to this purpose; they refer to the three cases shown in Figure 1, and in each case the load P is the only one acting:

Case I. Equation (3) gives the deflection at the corner:

$$z_c=\left(1.1-0.88\frac{a_1}{l}\right)\frac{P}{kl^2} \quad (13)$$

Case II. The deflection under the center of the load differs only slightly from the following value which is accurate when $a=0$:

$$z_i=\frac{P}{8kl^2} \quad (14)$$

Case III. The deflection at the point of application of a concentrated force P at the edge is approximately equal to

$$z_e=\frac{1}{\sqrt{6}}\,(1+0.4\,\mu)\,\frac{P}{kl^2} \quad (15)$$

that is, for $\mu=0.15$,

$$z_e=0.433\,\frac{P}{kl^2} \quad (16)$$

The quantity kl^2 occurring in each of these formulas may be expressed, according to equation (1), as

$$kl^2=\sqrt{\frac{Eh^3k}{12(1-\mu^2)}} \quad (17)$$

When experimental values of the deflections are at hand, one may determine the corresponding values of kl^2 by means of equations (13) to (16). Then equation (17) gives the value of k as

$$k=\frac{12(1-\mu^2)(kl^2)^2}{Eh^3} \quad (18)$$

Figures 4 to 11 are diagrams of deflections and moments. The titles of these figures explain the nature of the diagrams. The deflections and bending moments have been computed by means of the ordinary theory of slabs. The diagrams, therefore, give information concerning deflections in general, and concerning bending moments except in the immediate neighborhood of the concentrated load which produces the bending moments.

The diagrams in Figure 4 and Figure 5 have been obtained by an analysis which rests essentially on that given by the physicist Hertz [8] in 1884.

DETERMINATION OF DEFLECTIONS DUE TO MORE THAN ONE WHEEL

The diagrams in Figures 4 and 5 may be used in the following way for the purpose of finding the resultant deflections and stresses due to the combined influence of two or four wheel loads, each acting at a considerable distance from the edges of the slab.

Let each load be 10,000 pounds and let the horizontal rectangular coordinates of the centers of the four loads be as follows:

Coordinate	Load No. 1	Load No. 2	Load No. 3	Load No. 4
$x=$	0	66 in.	0	66 in.
$y=$	0	0	66 in.	66 in.

Loads 1 and 2 alone may represent the two rear wheels of a four-wheel truck, and the four loads combined may represent the four rear wheels of a six-wheel truck.

With $h=7$ inches, $E=3{,}000{,}000$ pounds per square inch, $\mu=0.15$, and $k=50$ lb./in^3., one finds by equations (1) and (17), or by Table 1:

$l=36.40$ inches; $kl^2=66{,}200$ pounds per inch; distances 1–2 and 1–3: 66 in.$=1.813l$; distance 1–4: $66\sqrt{2}=2.564l$. Then equation (14) as well as Figure 4 gives the following value of the deflection at point 1 due to Load No. 1:

$$z_{1,1}=\frac{P}{8kl^2}=\frac{10{,}000}{8\times 66{,}200}=0.0189 \text{ inch.}$$

Furthermore, Figure 4 leads to the following value of the deflection at point 1 due to load No. 2 alone:

$$z_{1,2}=0.03921\,\frac{P}{kl^2}=0.03921\,\frac{10{,}000}{66{,}200}=0.0059 \text{ inch.}$$

Then, by superposition of the two deflections, one finds the deflection at point 1 due to the combined influence of the two rear wheels 1 and 2:

$$z_{1,(1,2)}=z_{1,1}+z_{1,2}=0.0248 \text{ inch.}$$

The deflection at point 1 due to load No. 3 alone is

$$z_{1,3}=z_{1,2}=0.0059 \text{ inch.}$$

[8] H. Hertz. See "Über das Gleichgewicht schwimmender elastischer Platten," Wiedemann's Annalen der Physik und Chemie, v. 22, 1884, pp. 449–455; also in his Gesammelte Werke, v. 1, pp. 288–294. Hertz dealt with the problem of a large swimming slab, for example, of ice, loaded by a single force. A. Föppl, in his Technische Mechanik, v. 5, 1907, pp. 112–130, presented Hertz's theory in a modified, and in some ways simplified form, and he called attention to the applicability of this analysis to the problem of the slab on elastic support. Hertz made use of Bessel functions in his analysis. Since his analysis was published, the number of published numerical tables of Bessel functions has been increased. Among the newer tables those representing Hankel's Bessel functions,

$$H_0^{(1)}(x\sqrt{i}) \text{ and } H_1^{(1)}(x\sqrt{i}),$$

are of especial interest for the present problem. Tables of these functions may be found in the book of tables by E. Jahnke and F. Emde, "Funktionentafeln mit Formeln und Kurven," 1909, pp. 139 and 140. By means of these tables the numerical values given in Figures 4 and 5 were obtained by simple computations. Since these diagrams were prepared, two papers have appeared in which the same functions are used for the purpose of analysis of slabs on elastic support. One is by J. J. Koch, "Berekening van vlakke platen, ondersteund in de hoekpunten van een willekeurig rooster," De Ingenieur, 1925, No. 6; the other is by Ferdinand Schleicher, "Über Kreisplatten auf elastischer Unterlage," Festschrift zur Hundertjahrfeier der Technischen Hochschule Karlsruhe, 1925.

The deflection at point 1 due to load No. 4 alone is, according to Figure 4:

$$z_{1,4}=0.01620\frac{P}{kl^2}=0.0024 \text{ inch.}$$

By superposition of the four deflections due to each separate load, one finds the resultant deflection due to the four loads:

$$z_{1,(1,2,3,4)}=0.0331 \text{ inch.}$$

For the purpose of computing the state of stresses at the bottom of the slab under the center of load No. 1, it will be assumed that load No. 1 is distributed uniformly over the area of a circle with radius $a=6$ inches. The stresses due to load No. 1 will be the same in all directions, and they are, according to Table 3:

$$\sigma_x=\sigma_y=279 \text{ pounds per square inch.}$$

According to Figure 5, load No. 2 produces a radial bending moment, M_r, in this case in the direction of x, equal to $M_x=-0.0211P=-211$ inch-pounds per inch (or -211 pounds), and a tangential bending moment M_t, in this case in the direction of y, equal to

$$M_y=0.0181P=181 \text{ pounds.}$$

The corresponding stresses are found by dividing these bending moments by the section modulus per unit of width, that is, by $\frac{1}{6}h^2=8.167$ in.^{2}. Thus one finds the stresses in the directions of x and y:

$$\sigma_x=-\frac{211}{8.167}=-26 \text{ pounds per square inch}$$

and

$$\sigma_y=\frac{181}{8.167}=22 \text{ pounds per square inch.}$$

These stresses are principal stresses, that is, one is the maximum, the other the minimum stress, and there are no shearing stresses in the directions of x and y.

For the case of the four-wheel truck, one finds, then, by superposition, the following principal stresses due to the two rear wheels, loads No. 1 and No. 2, these principal stresses being in the directions of x and y:

$$\sigma_x=279-26=253 \text{ pounds per square inch.}$$

$$\sigma_y=279+22=301 \text{ pounds per square inch.}$$

STRESSES DUE TO SIX-WHEEL TRUCK

In the case of the six-wheel truck the effects of loads No. 3 and No. 4 must be included. Load No. 3 contributes the same stresses at point 1 as does load No. 2, only the indices x and y are to be interchanged. Consequently the resultant stresses in the directions of x and y due to the combined influence of loads 1, 2, and 3 become

$$\sigma_x=\sigma_y=279-26+22=275 \text{ pounds per square inch.}$$

These stresses, again, are principal stresses. Since they are equal, the horizontal stresses will be the same in all directions, each stress being a principal stress.

Let x', y' be a new system of horizontal rectangular coordinates with the axis of x' along the diagonal line from point 1 to point 4. Load No. 4 produces a radial bending moment in the direction of x' and a tangential bending moment in the direction of y'. According to Figure 5 these bending moments are

$M_{x'} = -0.0186P = -186$ pounds and $M_{y'} = 0.0058P = 58$, pounds respectively. The corresponding stresses are found, again, by dividing the bending moments by the section modulus per unit of width, that is, by 8.167 in.^{2}, and they are

$$\sigma_{x'} = -23 \text{ pounds per square inch and } \sigma_{y'} = 7 \text{ pounds per square inch.}$$

These stresses are principal stresses. The resultant principal stresses due to all four loads combined, therefore, are in the directions of x' and y', and have the values

$$\sigma_{x'} = 275 - 23 = 252 \text{ pounds per square inch.}$$

$$\sigma_{y'} = 275 + 7 = 282 \text{ pounds per square inch.}$$

One may draw the conclusion that the main part of the state of stresses at a given point is due to a wheel load right over the point. In the case examined, the contribution due to the three additional rear wheels of the six-wheel truck is of less importance than that due to the one additional rear wheel of the four-wheel truck.

Figure 6 and Figure 7 show deflections due to two wheel-loads combined. Each of these diagrams was obtained by superposition of two diagrams such as shown in Figure 4.

Figures 8 to 11 show effects of loads at the edge, but at a considerable distance from any corner.[9]

By virtue of Maxwell's theorem of reciprocal deflections, the deflection at a point B of any slab due to a load P at the point A is the same as the deflection at A due to a load P at point B. Figures 8 and 9 may be interpreted, therefore, in a double manner: First, as diagrams of deflections at any point B due to a load P at the particular point A at the edge; secondly, as influence diagrams, showing the deflection at the particular point A at the edge due to a load P at any point.

From this reciprocity of deflections one may draw a further conclusion which may be applied to Figures 8 and 9, and which concerns the curve of deflections, or elastic curve, which is obtained by intersection of the deflected middle surface by a vertical plane. Two lines L_A and L_B are drawn parallel to two opposite parallel edges of a slab. Two equal loads are considered, one acting at a point A of the line L_A, the other acting at a point B of the line L_B. The points A and B are assumed to be sufficiently far from the remaining two edges of the slab to permit the assumption of zero deformations at these edges. Then one may conclude that the elastic curve produced along the line L_B under the influence of the load P at A has exactly the same shape as the elastic curve produced along the line L_A under the influence of the load P at point B. In applying this conclusion to Figure 8 or Figure 9, let the line L_A be the edge shown on the drawing, and let the line L_B be at some distance from the edge. By the direct use of the diagrams one obtains the elastic curve at any line L_B parallel to the edge, due to a load at the edge. But one may interpret this curve as the elastic curve for the edge produced under the influence of a load at a point of the line L_B. The curvature of the deflected middle surface at point A of the edge in the direction of the edge, produced by the load P at any point B at some distance from the edge, is the same, accordingly, as the curvature of the deflected middle surface at point B in a direction parallel to the edge, as obtained in Figure 8 or Figure 9, due to the load P at the point A of the edge.

Thus Figures 8 and 9 may be used in studying the stresses produced along the edge by a wheel load at some distance from the edge.

The following use of the tables and diagrams is suggested. Let it be assumed that a certain pavement has been proved by tests and experience to be satisfactory for a given type of traffic. By the tables and diagrams one may compute, then, the corresponding critical stresses. These stresses may be adopted for the time being as allowable working stresses. With the stresses given, the tables and diagrams, through computations of the kind which has been shown, furnish answers to two questions: (1) What additional thicknesses are required if the wheel pressures are increased in a given manner; and, (2) what may be saved in the thicknesses by eliminating some of the heaviest vehicles. Prof. T. R. Agg has called attention to the importance of having an answer to the latter question, when one attempts to apportion the cost of the pavement to the various kinds of traffic for which it is used.

In using the tables and diagrams it should be kept in mind that the analysis is based on the assumptions which were stated at the beginning of this discussion. By the nature of these assumptions certain influences were left out of consideration, especially the following: (1) Variations of temperature, and other causes for tendency to change of volume; (2) the gradual diminishing of the thickness from the edge toward the interior; (3) local soft or hard spots in the subgrade; (4) horizontal components of the reactions of the subgrade; and (5) the dynamic effect, expressed in terms of the inertia of the pavement and subgrade. The horizontal components of the reactions of the subgrade, which are due to friction, may have a strengthening influence, especially at some distance from the edges, by causing a dome action in the pavement. As to the dynamic effects, with known values of the maximum pressure developed between the tire and the pavement, the effect of the inertia of the pavement may possibly be expressed approximately in terms of an increased value of the modulus, k. These additional influences are suitable subjects for further analysis.

[9] The theory by which these diagrams were obtained may be found in a paper by the writer: "Om Beregning af Plader paa elastisk Underlag med særligt Henblik paa Spørgsmaalet om Spændinger i Betonveje," Ingeniøren (Copenhagen), v. 32, 1923, pp. 513–524. See also "Die elastischen Platten," by A. Nádai, (Berlin) 1925, p. 186.

TESTS OF VIBROLITHIC CONCRETE

BY THE DIVISION OF TESTS, U. S. BUREAU OF PUBLIC ROADS

Reported by L. W. TELLER, Engineer of Tests, and C. E. PROUDLEY, Assistant Engineer of Tests

THE Bureau of Public Roads, cooperating with the American Vibrolithic Corporation has recently completed 28-day tests on concrete slabs constructed of the same materials in accordance with ordinary methods and the Vibrolithic process patented by the company.

The tests were made for the purpose of obtaining data on the relative strength of specimens equivalent in every particular except the method of construction. To that end every effort was made to eliminate all variables except the methods of placing, tamping, and finishing, and to have all operations performed under as nearly similar working conditions as possible.

Although it is recognized that the ultimate problem is one of economy, the current investigations have been confined to the study of certain physical properties of slabs made by the two methods; among them, the tensile strength as determined by bending, density, coefficients of expansion, and uniformity of the product.

The program of tests includes, in addition to those which have been made at the end of 28 days, others that will be made after one year; and the formulation of definite conclusions must await the completion of the latter tests. There are certain indications, however, which may be noted at this time with the understanding that they may be modified by the data obtained from the remaining tests.

Giving due consideration to all features of the investigation, such as the workability of the concrete and the method of finishing the normal specimens the following tentative conclusions may be drawn from the bending tests at the age of 28 days.

1. The Vibrolithic process resulted in a more uniform product.
2. For a given cement content, the slabs constructed by the Vibrolithic method exhibited greater strength than the normal concrete.
3. The strength of the slabs constructed by the Vibrolithic process, when tested with tension in the bottom, was practically the same as when tested with tension in the top.
4. The strength of the normal concrete, when tested with tension in the bottom, was less than when tested with tension in the top.

DIMENSIONS AND CONSTRUCTION OF THE TEST SLABS

The 36 by 72 inch slabs of each class were constructed of the same kinds of materials under conditions as nearly uniform as practicable. Designed to be 6 inches in depth, the individual slabs actually varied slightly from this dimension as indicated in Tables 1 and 2.

The slabs were constructed on a specially prepared subgrade which was sprinkled and rammed thoroughly with a 20-pound tamper the day before placing the concrete and sprinkled again the morning the slabs were poured. The 2 by 6 inch oiled, dressed-lumber outside forms set on the subgrade thus prepared inclosed a row of five slabs which were formed by 2 by 4 inch separators, the beveled upper edges of which were set 2⅛ inches below the top of the outside forms, as shown in Figure 1.

The cement used was a brand of known reputation and predetermined satisfactory quality. All cement for a day's run was thoroughly mixed and resacked in lots of 47 pounds each and one of these sacks was tested each day in the laboratory with the results shown in Table 3 and Figure 12. Other physical properties of the cement were as follows:

Specific gravity	3. 10
Fineness, retained on 200-mesh sieve .... per cent	15. 1
Soundness	O. K.
Time of set (Gilmore needle):	
Initial	3 hrs. 25 min.
Final	6 hrs. 55 min.
Normal consistency .... per cent	23. 0

The aggregates used in all slabs—Potomac River sand and limestone in two sizes obtained from Frederick, Md.—had the following grading and physical properties.

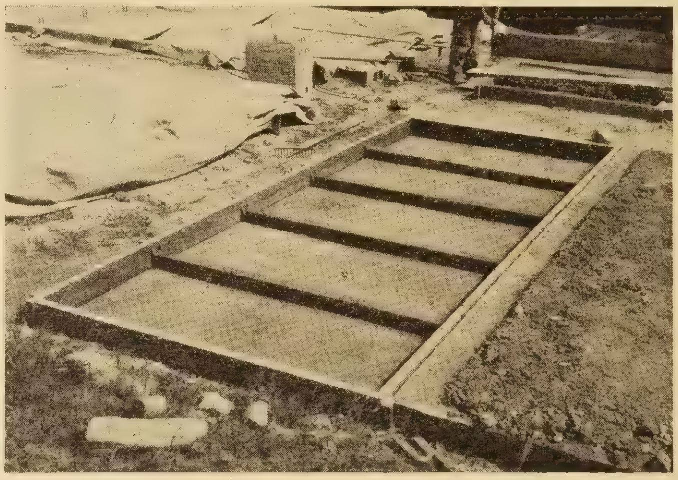


FIG. 1.—Forms and separators in place

GRADING AND PHYSICAL PROPERTIES OF THE SAND

Passing ¼-inch screen	per cent	91
10-mesh sieve	do	76
20-mesh sieve	do	65
30-mesh sieve	do	47
40-mesh sieve	do	35
50-mesh sieve	do	20
100-mesh sieve	do	7
200-mesh sieve	do	4
Loss by washing	do	2. 9
Tensile strength ratio:		
7 days	do	114
28 days	do	118
Weight per cubic foot, dry	pounds	105
Organic matter		immaterial coloration.

Bulking with 4 to 6 per cent moisture, approximately 20 per cent.

GRADING AND PHYSICAL PROPERTIES OF THE STONE

	Small stone per cent	Large stone per cent
Passing 2-inch screen		98. 6
1½-inch screen	100. 0	64. 7
1-inch screen	84. 4	6. 0
¾-inch screen	42. 9	1. 2
½-inch screen	12. 3	. 2
¼-inch screen	1. 8	0
10-mesh sieve	1. 0	

Percentage of wear		4. 2
Hardness coefficient		17. 5
Toughness		17
Specific gravity		2. 70
Absorption	per cent	. 10
Weight per cubic foot, solid	pounds	168
Crushed:		
Small stone	do	95
Large stone	do	98
Mixed 50–50	do	100

In order to obtain a unit volume of the mixture in equal parts of large and small stone it was found that 12½ per cent additional volume of each was required; i. e., a mixture of 1⅛ cubic feet of each size yielded 2 cubic feet.

To compensate for the bulking of the sand and the oversize material in it, 1¼ cubic feet of damp sand was used volumetrically as the equivalent of 1 cubic foot of dry sand. The small quantity of coarse sand considered as of gravel size did not affect the volume of stone in the proportions used. Figure 2 shows the grading curves of the aggregates used.

One bag of cement weighing 94 pounds was assumed to be equal to 1 cubic foot; and cubic-foot boxes, carefully marked in quarters, were used for measuring the sand and stone. The volumes of materials actually used in each batch were as follows:

NORMAL CONCRETE

	Proportions			
	1 : 1½ : 3	1 : 2 : 3	1 : 2 : 3½	1 : 2 : 4
Cement, bags	1½	1½	1½	1
Sand (damp) cubic feet	2¾	3¾	3¾	2½
Stone:				
Large do	2½	2½	3	2¼
Small do	2½	2½	3	2¼

VIBROLITHIC CONCRETE

	Proportions			
	1 : 1½ : 3½	1 : 2 : 3½	1 : 2 : 4	1 : 2 : 4½
Cement, bags	1½	1½	1	1
Sand (damp) cubic feet	2¾	3¾	2½	2½
Stone:				
Large do	3	3	2¼	2½
Small do	3	3	2¼	2½

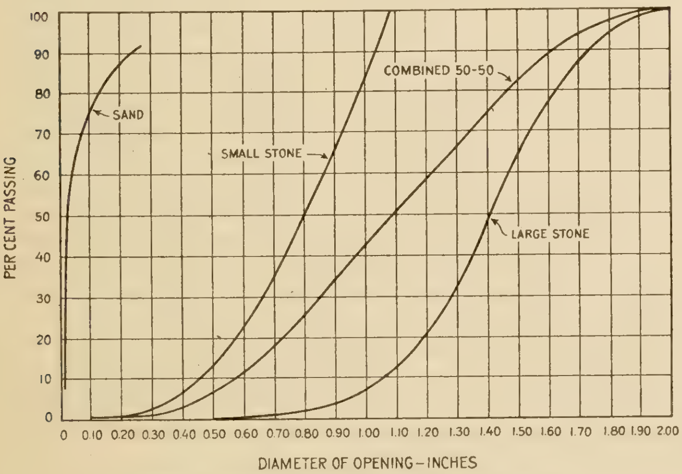


FIG. 2.—Grading of aggregates used in the concrete

The 10-cubic-foot, drum-type mixer, driven electrically at the rate of 20 r. p. m., was charged with the proper quantities of stone, sand, and cement in the order named, followed by sufficient clear Potomac River water to give the required consistency. The water tank was equipped with a gage glass calibrated to quarters of gallons. As the concrete was discharged from the mixer a flow determination was made on a portion of each batch, and as nearly as possible, the consistency was regulated so as to give a flow of 110 to 115 on the 30-inch flow table. As a further check on the consistency the second and sixth batches of every run were tested for slump with the approved slump cone, the requirement being a slump of about 2 inches.

FIG. 3.—Finishing Vibrolithic surface with steel float prior to belting

The concrete, which was placed in the forms without regard for the separators, was tamped and struck off with a straight 2 by 12 inch strike board worked lengthwise over each series of five slabs, and the desired finish of the normal concrete was obtained by belting lengthwise over the slabs with an 8-inch rubber belt.

The Vibrolithic sections were similarly placed and struck off, after which they were covered uniformly with Frederick limestone of 2-inch to 1-inch size at the following rates:

Mix	1:1½:3½	1:2:3½	1:2:4	1:2:4½
Pounds per square yard	50	50	45	40

On this stone were placed special racks over which the vibrators were run, according to the patented process. Approximately four minutes of vibration was allowed for each slab. The removal of the racks left an irregular mortar surface which was smoothed down with a long-handled steel float as shown in Figure 3, after which the surface was belted. Dry spots were sprinkled with a little water to facilitate finishing.

All slabs, as soon as their hardness would permit, were covered with wet burlap which was kept damp during the day and thoroughly wet down before leaving at night. The following morning it was removed and replaced with a covering of damp earth. The earth was kept damp by daily sprinkling until 28 days had elapsed, at which time the top surface was cleaned. After the removal of the earth the slabs were kept damp by means of wet burlap until time for testing. Slabs to be tested at one year will remain uncovered 10 months, but will again be covered with damp earth for the 28 days immediately prior to testing.

TABLE 1.—*Summary of data of tests on normal concrete slabs*

Mix	Slab No.	Surface in tension	Depth	Section modulus, $\frac{I}{c}$	Total load at rupture	Modulus of rupture: Individual slabs	Modulus of rupture: Averages	Strength ratio = tension in top / tension in bottom	Variation of individual tests from average of groups
			Inches		*Pounds*	*Pounds*	*Pounds*		*Per cent*
1:1½:3	86	Top	6.1	227.2	17,810	784			12.0
	88	do	6.2	238.0	18,010	757			8.2
	90	do	6.3	241.3	15,610	647	700		7.6 } 8.1
	97	do	6.4	253.1	17,660	698			.3
	99	do	6.5	260.7	16,010	614	} 624	1.275	12.2
	87	Bottom	6.0	218.4	13,960	[1] 551			.4
	89	do	6.2	233.6	12,410	532			3.1
	96	do	6.3	239.7	15,510	647	549		17.8 } 7.4
	98	do	6.4	248.3	12,560	506			7.8
	100	do	6.5	255.0	12,910	507			7.7
1:2:3	116	Top	6.1	227.2	14,460	636			5.0
	118	do	6.2	238.0	13,660	574			5.3
	120	do	6.1	227.2	14,310	630	606		4.0 } 5.7
	167	do	6.2	238.0	15,210	639			5.5
	169	do	6.2	238.0	13,160	553	} 595	1.039	8.8
	117	Bottom	6.1	225.5	13,160	584			.2
	119	do	6.0	218.4	10,960	502			13.9
	166	do	6.4	247.3	16,680	675	583		15.8 } 6.7
	168	do	6.2	233.6	13,220	566			2.9
	170	do	6.3	239.7	14,080	588			.9
1:2:3½	127	Top	6.3	245.4	14,360	585			2.5
	129	do	6.2	238.0	14,840	623			3.7
	136	do	6.5	256.6	15,560	607	600		1.2 } 5.2
	138	do	6.5	260.7	16,920	649			8.1
	139	do	6.7	277.3	14,910	538	} 586	1.047	10.3
	126	Bottom	6.4	247.3	13,760	557			2.8
	128	do	6.3	240.7	14,210	590			3.0
	130	do	6.5	255.0	14,860	583	573		1.7 } 2.8
	137	do	6.5	255.9	13,980	546			4.7
	140	do	6.6	262.1	15,260	582			1.6
1:2:4	147	Top	6.0	224.3	11,360	507			9.5
	149	do	5.9	216.4	10,510	486			13.2
	156	do	6.1	227.2	14,340	631	560		12.7 } 10.1
	158	do	6.2	238.0	15,000	630			12.5
	160	do	6.6	265.3	14,460	545	} 507	1.233	2.7
	146	Bottom	6.2	232.2	9,760	420			7.5
	148	do	5.9	212.0	8,920	421			7.3
	150	do	6.2	232.2	9,860	424	454		6.6 } 8.7
	157	do	6.3	240.7	12,720	529			16.6
	159	do	6.4	248.3	11,860	478			5.3
Mean								1.149	Top 7.5 Bottom 6.4

[1] Broke 17¼ inches from end support.

TABLE 2.—*Summary of data of tests on Vibrolithic concrete slabs*

Mix	Slab No.	Surface in tension	Depth	Section modulus, $\frac{I}{c}$	Total load at rupture	Modulus or rupture: Individual slabs	Modulus or rupture: Averages	Strength ratio = tension in top / tension in bottom	Variation of individual tests from average of groups
			Inches		*Pounds*	*Pounds*	*Pounds*		*Per cent*
1:1½:3½	82	Top	6.1	231.5	16,536	714			0.1
	84	do	6.2	238.0	16,286	684			4.3
	91	do	6.1	227.2	16,386	721	715		0.9 } 4.1
	93	do	6.2	238.0	16,036	674			5.7
	95	do	6.1	227.2	17,786	783	} 708	1.020	9.6
	81	Bottom	6.1	224.2	16,486	736			5.0
	83	do	6.2	233.6	17,086	732			4.5
	85	do	6.1	224.2	14,036	626	701		12.1 } 4.8
	92	do	6.0	218.4	15,586	714			1.8
	94	do	6.1	225.5	15,686	696			0.7
1:2:3½	112	Top	5.9	216.4	14,336	662			3.4
	114	do	6.1	231.5	13,986	604			11.8
	161	do	6.2	233.8	18,106	774	685		13.0 } 7.1
	163	do	6.1	231.5	16,586	717			4.7
	165	do	6.2	233.8	15,566	666	} 669	1.047	2.8
	111	Bottom	6.3	239.7	16,886	705			7.8
	113	do	6.2	233.6	15,136	648			0.9
	115	do	6.4	247.3	13,336	539	654		17.6 } 7.4
	162	do	6.4	248.3	16,636	670			2.5
	164	do	6.3	240.7	16,986	706			8.0
1:2:4	121	Top	6.4	248.9	15,006	603			2.6
	123	do	6.3	245.4	14,286	582			1.0
	125	do	6.5	256.6	15,136	590	588		0.4 } 1.9
	132	do	6.2	238.0	14,236	598			1.7
	134	do	6.1	231.5	13,126	567	} 579	1.030	3.6
	122	Bottom	6.3	240.7	13,026	542			5.1
	124	do	6.2	233.6	13,586	582			2.0
	131	do	6.5	255.0	14,406	565	571		1.1 } 4.1
	133	do	6.3	240.7	14,866	617			8.1
	135	do	6.2	232.2	12,726	548			4.0
1:2:4½	141	Top	6.3	241.3	12,986	538			5.9
	143	do	6.0	224.3	12,646	564			1.4
	145	do	6.1	227.2	13,986	615	572		7.5 } 5.6
	152	do	6.3	245.4	13,086	533			6.8
	154	do	6.3	245.4	14,960	610	} 578	0.981	6.6
	142	Bottom	6.0	218.4	13,306	610			4.7
	144	do	6.2	233.6	11,586	496			14.9
	151	do	6.3	239.7	15,586	651	583		11.7 } 7.4
	153	do	6.3	240.7	13,566	563			3.4
	155	do	6.6	262.1	15,646	597			2.4
Mean								1.020	Top 4.7 Bottom 5.9

THE METHOD OF TESTING

When the slabs were ready to be tested they were lifted from the subgrade by means of a cradle of the special design shown in Figure 4. As, in nearly every case, cracks had formed over the separators there was no difficulty in raising the individual slabs; but in most instances a large amount of subgrade came up with the slabs and had to be removed.

Half of the slabs were tested with tension in the top surface, the others with tension in the bottom, the former condition being effected by inverting the slab in the testing machine. In order to obtain a solid bearing for the knife edges on the under surface of the slabs, flat steel strips 1¼ inches wide and three-sixteenths inch thick were set in plaster of Paris in the proper position. These strips were not needed on the finished surface, as the rubber pads on the knife edges, shown in Figures 5 and 6, took up the slight irregularities which, in general, were merely the marks of the float or belt and were not more than one-sixteenth inch deep.

Referring to Figure 7, which shows the elevation of the testing machine, the method of testing may be described as follows: The slab, *A*, was carefully centered on the lower knife edge, *B*, and the rocking knife edge, *C* (see also fig. 5). The upper knife edges, *D*, mounted with the hydraulic jack, *E*, and the calibrated head, *K*, on the carrier plate, *G*, was swung into position over the slab so that loading was at the third points of the slab. The collar, *I*, kept the carrier plate elevated so that the knife edges would swing clear of the slab until in position, at which time the screw jack, *J*, was used to lower guide post, *H*, to which the collar was fastened. The collar was lowered until the carrier plate was entirely free. The follower head, *L*, carried by the plate, *M*, was then swung about guide post, *H*, and brought into contact with the top of the machine by means of hand screw, *N*. The initial load was then put on the slab by means of the jack handle of hydraulic jack, *E*, after which the load was continuously and steadily applied by means of the small auxiliary pump, *F*. The calibrated beams, Figure 8, were arranged to deflect as simple beams as the load was applied. The dial, read initially before the load was applied, was watched carefully by at least two observers so as to catch the maximum dial reading, and a calibration chart was used for converting the difference in dial readings to total applied load in pounds.

FIG. 4.—Lifting slab from subgrade

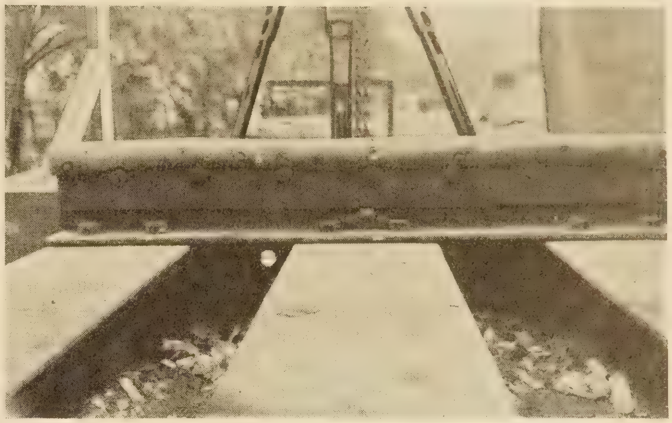


FIG. 5.—Lower knife edge showing rocker pin and rubber pad

The broken sections were removed and stacked with their broken faces outward, as shown in Figures 10 and 11, and their cross sections were then measured by two operators each of whom made four depth measurements on each slab.

COMPUTATION OF MODULUS OF RUPTURE

The weight of the slab itself, the weight of the upper knife edges and loading device, and the pressure applied by the jack comprised the total load. The uniformly distributed weight of the slab was converted into a concentrated load which could be added to the weight on the knife edges. Thus, the weight of the 1:2½:3 normal concrete slabs, 152 pounds per cubic foot, was found to be equivalent to two loads of 410 pounds each concentrated at the knife edges; and the 1:1½:3½ vibrolithic slabs which weighed 157 pounds per cubic foot were equivalent to two concentrated loads of 424 pounds. The dead load of the knife edges, jack, etc., was found to be 640 pounds, or at each knife edge, 320 pounds.

From an initial load computed in this manner the pressure on the slabs was gradually increased by means of the hydraulic jack, *E* (fig. 7), until rupture occurred, at which time the pressure indicated by the calibrated beams added to the initial dead load constituted the total load, *P*.

The extreme fiber tensile stress, S, was then computed from the formula

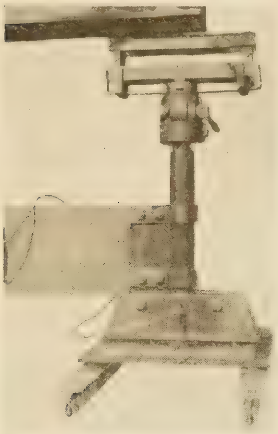


Fig. 6.—Arrangement of loading knife edges

$$S = M\frac{c}{I}$$

in which the bending moment for the 60-inch span with third-point loading is

$$M = \frac{P}{2} \times 20 = 10P.$$

The section modulus, $\frac{I}{c}$, computed from the measurements of the broken sections, was found to be different in almost every specimen, principally because of variations in the thickness of the slabs and in the location and number of the separator projections on the edges. The projections, or sections of the slabs formed over the intermediate separating forms, were trapezoidal in cross section, approximately seven eighths inch in width and from 2⅛ to 3 inches in depth.

The thicknesses of the several slabs, their section moduli, the total loads at rupture, and the computed moduli of rupture are shown for the two classes of concrete in Tables 1 and 2, respectively. The moduli of rupture are shown also in Figure 12 and Table 3, with other factors indicative of the characteristics of the concrete.

These other factors include the tensile strength of 1:3 mortar briquettes made of the cement used in the slabs and Ottawa sand; the modulus of rupture of 1.2 Ottawa sand mortar beams; the consistency of the concrete as indicated by its flow; the water-cement ratio; and the ratio of the cement to aggregate by weight for each of the slabs.

The water-cement ratio is a volumetric relation and assumes 1 bag of cement to equal 1 cubic foot. Thus, if a batch of concrete were mixed with 7½ gallons of water for each bag of cement, the water-cement ratio, $\frac{W}{c}$, would be recorded as 1.00. In computing the relation the quantity of moisture contained in the sand, as found by a daily moisture determination, was

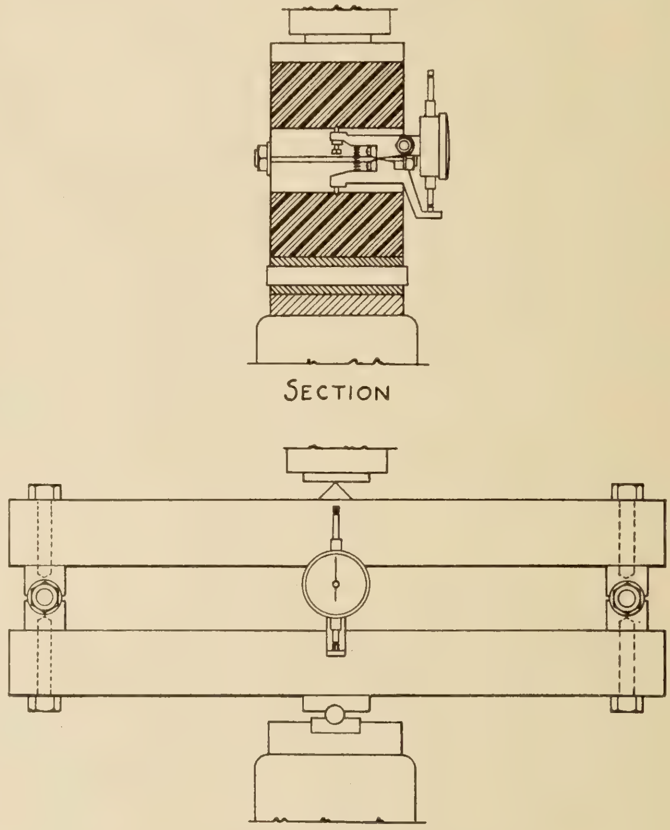


Fig. 8.—Calibrated beams used to measure the load

added to the volume of water introduced in mixing to obtain the total value of W.

The cement-aggregate ratio is based on the nominal mix for both Vibrolithic and normal concrete. The top stone of the Vibrolithic concrete is not included

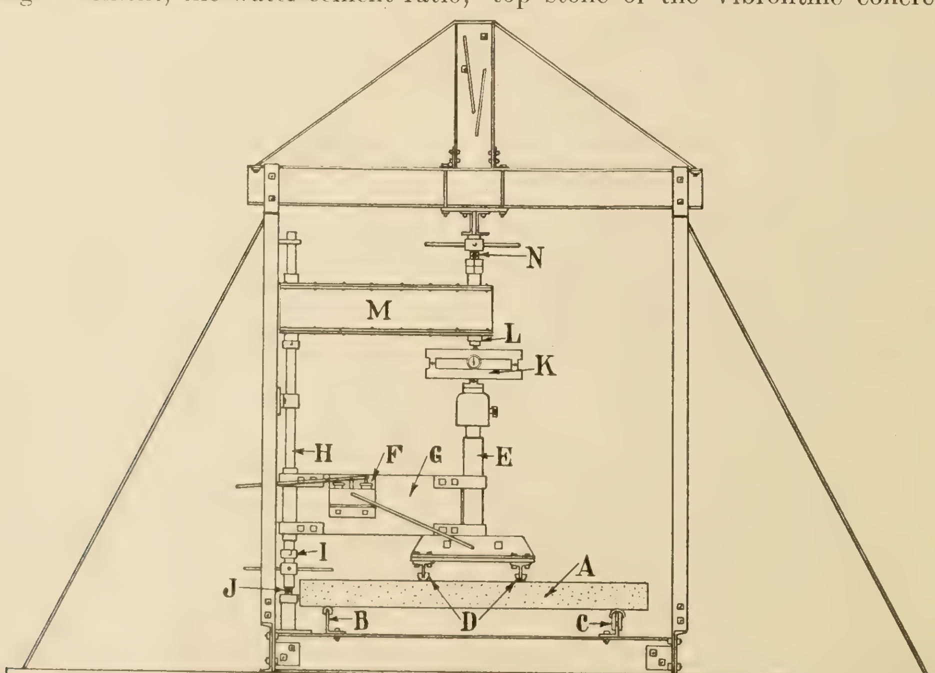


Fig. 7.—Apparatus for testing the slabs

with the aggregate in computing the relation. Using the weights per cubic foot of the sand and stone as determined by tests, the volumetric proportions were readily converted to weights; and the weight of cement divided by the total weight of sand and stone gives the ratio.

CEMENT-AGGREGATE AND WATER-CEMENT RATIOS AS STRENGTH INDICES

Past experience naturally leads to the expectation that the strength of concrete will follow, in general, the cement content. Table 3 and Figures 9 and 12 show that the expected relationship obtained in these tests, the cement content being expressed as the ratio of cement to total aggregate in the mix.

The water-cement theory is also seen to agree with the strengths of the slabs. The leaner mixes were gauged with higher water-cement ratios and consequently gave lower strength. It is seen, however, that there is practically no difference between the average strength of the 1:2:3 and the 1:2:3½ normal concrete, although the other data do not show the reason for this.

Similarly, there is no difference between the average strength of the 1:2:4 and the 1:2:4½ Vibrolithic concrete. In this case, however, the $\frac{W}{c}$ is the same for each of these proportions. This probably accounts, in part at least, for the uniformity in strength. It is also interesting to note in this connection that the quality of the cement, as indicated by the tension tests of mortar, was lower for that used in the 1:2:4 mix than in the 1:2:4½ mix, whereas the quality as indicated by the cross-bending test of 1:2 Ottawa sand mortar beams is the reverse. It is probable that a more thorough examination of the slabs as called for by the program of tests will furnish information that will explain some of these deviations.

The relation between the quantity of cement used and the strength obtained is shown in Figure 9. For equivalent quantities of cement the strength of the Vibrolithic concrete is higher than the normal concrete. The difference amounts to about 16 per cent for 1:2:3 and 1:1½:3½ concrete, and about 21 per cent for 1:2:4 concrete based on the strength of the normal concrete.

Considering the two processes from the standpoint of the cement required to give equal strength, a comparison at 600 pounds per square inch modulus of rupture shows that roughly, 19 per cent additional cement is required in normal concrete; or, in other words, only 84 per cent of the cement required for normal concrete is necessary for equal strength in Vibrolithic work. At higher strengths the curves (fig. 9) indicate slightly greater advantages for the Vibrolithic, and at lower strengths less advantage insofar as saving in cement is concerned.

TOP AND BOTTOM STRENGTH OF VIBROLITHIC SLABS NEARLY EQUAL

Due to difference in density and, possibly, to other variables as yet undetermined, the strength obtained when the bottom of the slab was in tension was, in most cases, lower than when the top was tested in tension. The average amount of this difference is shown graphically in Figure 12, the shaded bars giving the strength with the top in tension and the solid bars the strength with the bottom in tension. This is also shown in Tables 1 and 2.

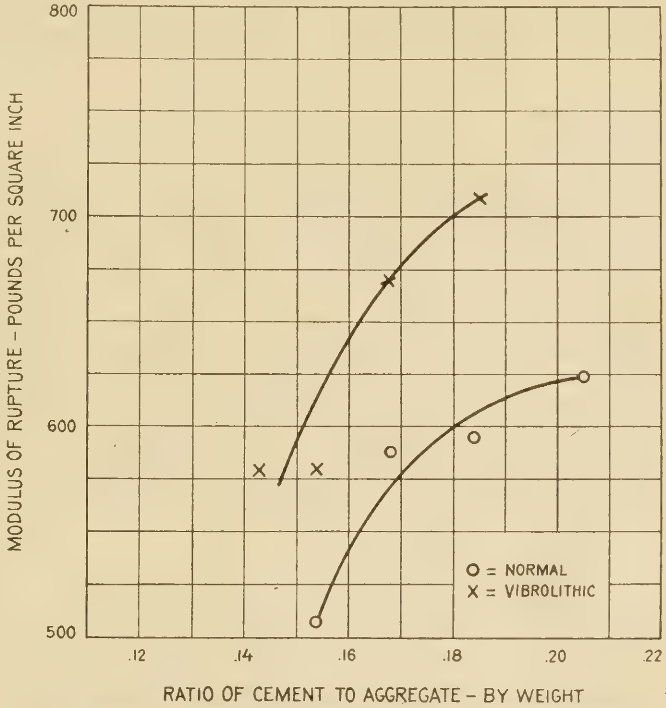


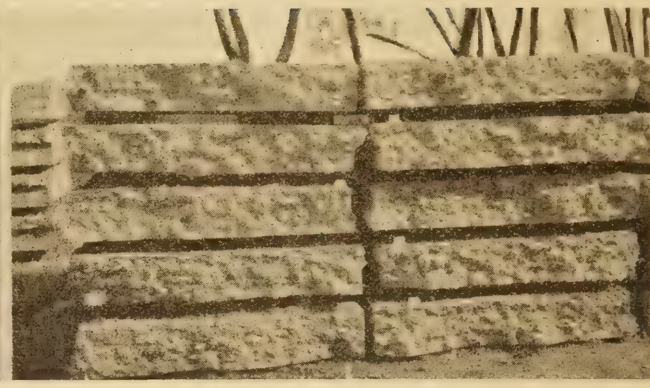
FIG. 9.—Relation of modulus of rupture of normal and Vibrolithic concrete to cement-aggregate ratio

It has been suggested that, as a result of excessive tamping and finishing, the strength of the upper surface of a concrete road might be increased beyond that of the bottom. These tests do not substantiate this theory. There is no doubt that the Vibrolithic method is the more vigorous finishing treatment; nevertheless, insofar as the uniformity of strength in top and bottom of the slab is concerned, the Vibrolithic is more remarkable than the normal concrete. Expressed numerically, the average resistance to tension in the bottom of normal concrete slabs is 87.7 per cent of that in the top, and for Vibrolithic it is 98.0 per cent.

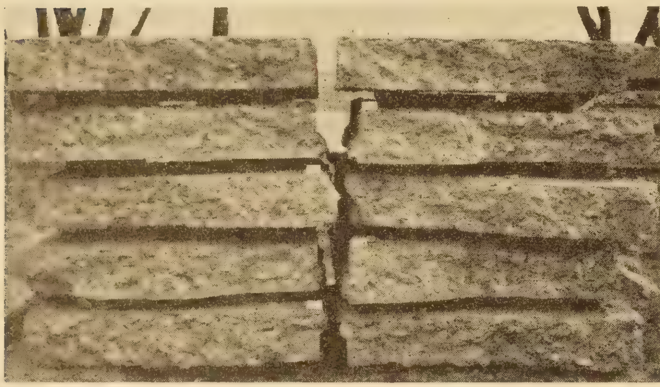
TABLE 3.—*Results of tests of slabs, aggregate, and cement*

Finish	Normal concrete								Vibrolithic concrete							
Mix	1 : 1½ : 3		1 : 2 : 3		1 : 2 : 3½		1 : 2 : 4		1 : 1½ : 3½		1 : 2 : 3½		1 : 2 : 4		1 : 2 : 4½	
Slab numbers, inclusive	86 to 90	96 to 100	116 to 120	166 to 170	126 to 130	136 to 140	146 to 150	156 to 160	81 to 85	91 to 95	111 to 115	161 to 165	121 to 125	131 to 135	141 to 145	151 to 155
Modulus of rupture concrete:																
Tension in top	729	656	613	596	604	598	496	602	699	726	633	719	592	582	572	571
Tension in bottom	542	553	543	610	577	564	422	503	698	705	631	688	562	577	553	604
Ratio of cement to aggregate by weight	0.205		0.184		0.168		0.154		0.185		0.168		0.154		0.143	
Water-cement ratio	0.67	0.73	0.78	0.77	0.88	0.89	0.86	0.86	0.69	0.75	0.80	0.80	0.91	0.91	0.91	0.90
Consistency flow table	112	112	114	112	111	112	109	108	110	113	110	109	107	107	114	112
Modulus of rupture 1:2 mortar	775	811	815	707	775	748	655	753	775	811	815	707	775	748	655	753
Tensile strength 1:3 mortar briquettes	385	335	355	355	345	380	385	395	385	335	355	355	345	380	385	395

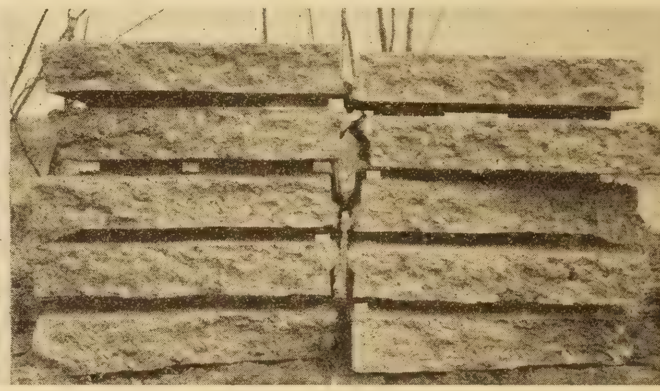
1:1½ : 3½ mix



1:1½ : 3½ mix



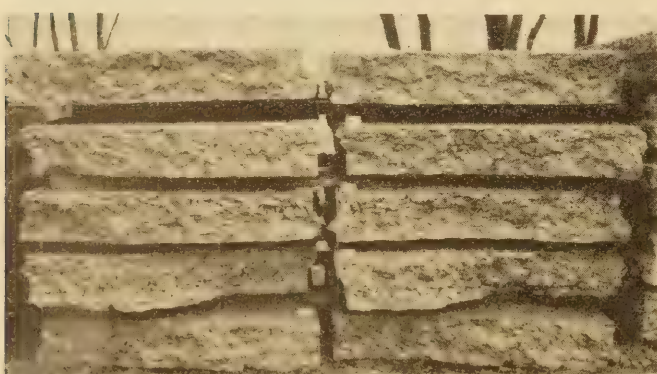
1: 2 : 3½ mix



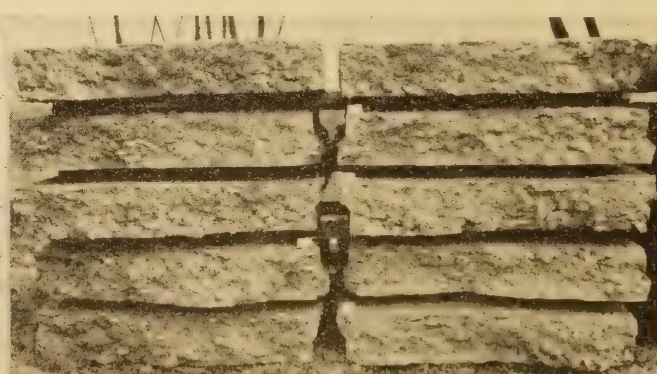
1: 2 : 3½ mix

1: 2 : 4 mix

1: 2 : 4 mix

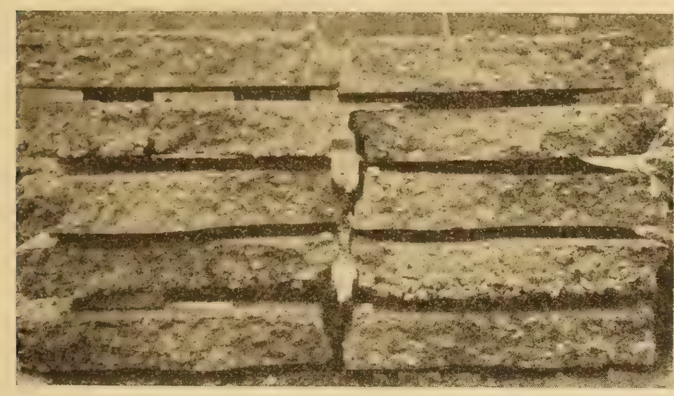


1: 2 : 4½ mix



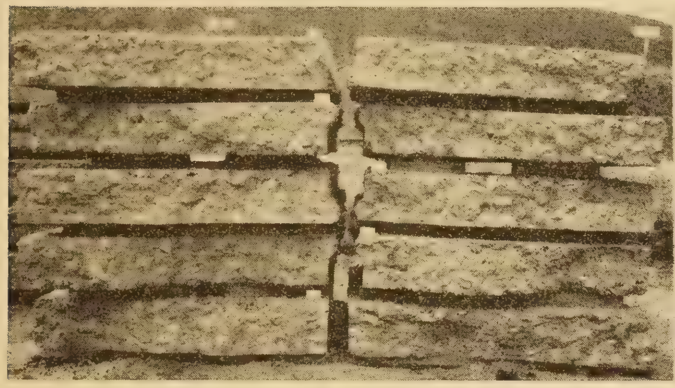
1: 2 : 4½ mix

FIG. 10.—Vibrolithic concrete slabs of various mixes showing cross sections at point of failure in test



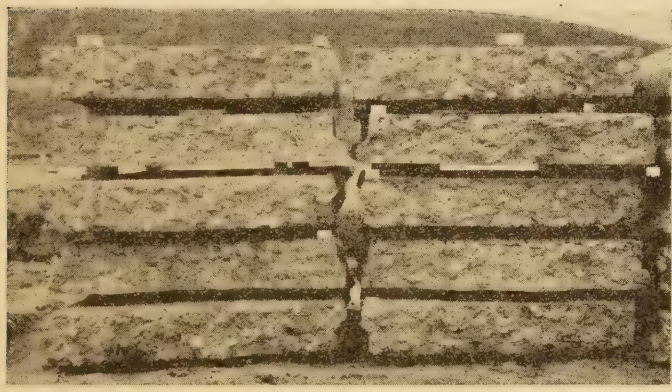
1: 1½ : 3 mix

1: 1½ : 3 mix



1: 2 : 3 mix

1: 2 : 3 mix



1: 2 : 3½ mix

1: 2 : 3½ mix

1: 2 : 4 mix

1: 2 : 4 mix

Fig. 11.—Normal concrete slabs of various mixes showing cross sections at point of failure in test

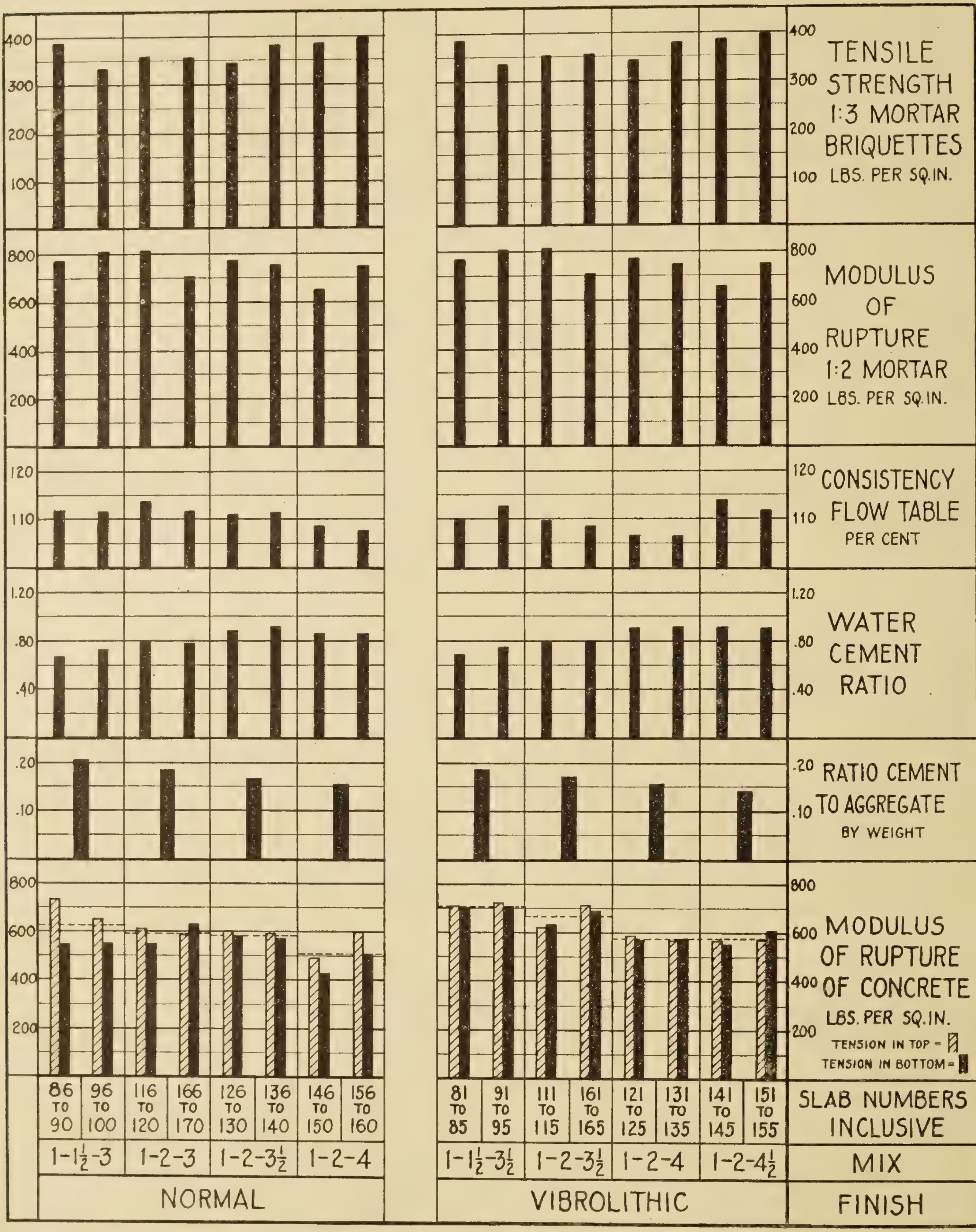


FIG. 12.—Results of tests of normal and Vibrolithic concrete and mortar briquettes and beams

Another important consideration not to be overlooked is that of uniformity of the product. It may be seen in the last column of Tables 1 and 2 that the average variation of the individual specimens from the average for the five specimens of the group is a little less in the case of the Vibrolithic than in the normal concrete. The average of all of these variations for Vibrolithic specimens is 5.3 per cent, and for the normal concrete specimens 6.8 per cent.

The photographs, shown in Figures 10 and 11, furnish an explanation of some of the apparent abnormalities in the results given in Tables 1 and 2. It is reasonable to suppose that, other things being equal, the slab with the minimum void spaces will give greatest strength; also, that the closer voids are to the outer surfaces, the more serious will be the effect on the strength. The distribution of void spaces in the specimens under consideration may be seen by a careful inspection of the photographs.

The 1:1½:3½ Vibrolithic specimens, Figure 10, show very few voids. Slab 94, which contains more than any other of this mix and type, shows one group of voids near the bottom and another a little below the center; but their effect is apparently negligible.

The 1:2:3½ specimens of Vibrolithic concrete are also quite dense except for a thin layer of voids occurring in slab 162 about one inch from the bottom, which seems to have had no serious effect on the strength.

The specimens of 1: 2: 4 Vibrolithic concrete exhibit a greater percentage of air pockets due, no doubt, to the leaner mix. It is interesting to observe the position of the voids in slabs 124, 131, 135, and especially in 121 and 122, where they are quite close to the neutral axis.

The 1: 2: 4½ mix of the Vibrolithic series apparently incloses a still greater number of voids, particularly in slabs 142, 143, 144, 151, 153, and 155. In 142 and 143 they are principally in the compression side as tested and probably have little effect except as they tend to lower the neutral axis and thus shorten the distance to the extreme fiber. In slab 144 a bad condition exists near the bottom, which is reflected in the unusually low strength obtained. The same is also true of slab 153. Slab 155 shows considerable void space, at an average distance of about 1½ inches from the bottom.

The richest mix in the normal concrete slabs, shown in Figure 11, is 1: 1½: 3. The most noticeable voids in slabs of this mix occur in Nos. 87, 88, 89, 97, 98, and 100. Of these slabs, 88 and 97 were effected only in the compression side, but the other specimens mentioned are penetrated varying distances up to 2½ inches in the tension side. These conditions, no doubt, explain some of the unusually low strengths obtained in this group.

The next leaner mix, 1: 2: 3, also shows many voids extending from the bottom well into the slab. Slabs 116, 120, 168, and 170 are badly honeycombed in the tension side, and in 117 and 119 the voids extend so deeply into the compression side as to cause a decrease in effective depth and consequent decreased strength when based on the slab thickness as measured.

In the 1: 2: 3½ mix of normal concrete, slabs 126, 130, 137, and 140 are most notable for their honeycombing, although every slab of this group includes an appreciable quantity of voids. The 1: 2: 4 normal concrete is in particularly poor condition due, very likely, to the leanness and dryness of the mix. The consistency as measured by the flow table was no less workable, however, than much of the concrete used in present-day construction of first-class concrete pavement on which a machine finishing is used.

EFFECT OF FINISHING METHODS OF NORMAL SLABS

Special consideration should be given the method of constructing the normal concrete slabs for this investigation, as it is plainly upon this factor that the value of these tests depends.

The concrete for the normal specimens was dumped into the forms, shoveled into place, and spaded along the edges of the forms. The specimens were tamped across once with the 2 by 12 inch strike board, after which the surface was struck off and belted to a finish.

As the consistency was held to the same value used for the Vibrolithic specimens, it was more nearly that which should be used for machine finishing, and the hand finishing described was not adequate. This was not apparent until the specimens were turned over at the time of testing, but the result was an unfortunate amount of honeycombing and consequent variation in the test results.

It is believed that, if the concrete in the normal specimens had been more thoroughly compacted as would be the case in machine finishing, less variation in strength would have resulted and generally higher values would have been obtained. This is indicated by the fact that those specimens where the lack of compaction was most apparent showed noticeably lower resistance to cross-bending.

An interesting general observation regarding the location of the void spaces in the specimens thus far tested is to be made from an inspection of the photographs and much better, of course, by a study of the actual specimens. Through the action of the vibrator on the surface of the concrete in the Vibrolithic process, mortar is apparently worked down to the subgrade, thus leaving the air spaces at some distance from the bottom of the slab. When the ordinary methods of finishing are used the voids which occur at the bottom while the concrete is being deposited remain unfilled, as a result of the arch action of the coarse aggregate. Advantages of concrete made by the Vibrolithic method, as brought out in this report, may be traced in many instances to the more favorable location of voids.

TEMPERATURE AS A FACTOR IN THE STABILITY OF ASPHALTIC PAVEMENTS

BY THE DIVISION OF TESTS, U. S. BUREAU OF PUBLIC ROADS

Reported by W. J. EMMONS, Highway Research Specialist, and B. A. ANDERTON, Chemical Engineer

CERTAIN asphaltic paving mixtures, notably those containing excessively high percentages of bitumen for the accompanying aggregates, are susceptible to displacement under traffic. Such weakness is generally manifested during the warmer seasons of the year when the bitumen, present in space-filling volumes of considerable magnitude, absorbs sufficient radiant energy from the sun to render it fluid. The pavement then exists as a structure composed of a more or less independently stable aggregate and a viscous fluid of low binding power. When, under these conditions, the pavement is subjected to traffic of sufficient magnitude the resisting power of the plastic mass is soon overcome and the development of ruts and waves occurs.

FIG. 1.—General view of experimental track surfaced with asphaltic pavements showing central temperature measurement station and thermo-couple wires

During the past three years the Bureau of Public Roads has conducted a series of tests to measure the internal temperatures of asphaltic pavements and to determine in a general way the effect of such temperatures upon the resistance to displacement under traffic. These tests were conducted in connection with two series of stability experiments which involved the construction of 60 sections of pavement on a circular track or roadway 13 feet in width. The mixtures included coarse-graded asphaltic concretes and sheet asphalts and were laid 2 inches thick upon a very smooth concrete foundation.

For the purpose of determining their relative displacement under traffic, screws were driven into the pavement across the line of traffic and referenced to permanent markers set in the concrete. Thermocouples of copper and constantin wire were installed in many of the pavements and connected to a central station for the observation of temperatures. Most of the thermocouples were set in the pavements at a depth of one-half inch below the surface, although in several instances additional ones were placed at half-inch intervals down to the concrete foundation. Figure 1 shows the general location of the experimental pavements, the central temperature measurement station and the thermocouple wires leading to it. Observations of temperatures were made at 10 a. m. and 2 p. m. each day when the track was subjected to traffic. It was found that an average of the air temperatures at these times very closely approximated the average air temperature for the daily period of operation of the test. Moreover, the maximum air temperature during the day was recorded at approximately 2 p. m.

The first series of stability tests was made on 27 different coarse-graded asphaltic concrete mixtures. Construction was completed late in the year and the operation of a loaded 3-ton truck over the test sections was begun in October. During the autumn, winter, and early spring months the average air temperature at 2 p. m. was below 65° F. and all of the pavement mixtures remained rigid. Late in the following May a sudden and decided rise in temperature took place, maintaining for the following four weeks a 2 o'clock average of about 80° F.

The effect of this increased temperature was immediately evident. Although many mixtures remained entirely stable, a number began to shove and rut under the continuing traffic. This series of mixtures was subjected to 50,000 passages of the truck, of which 60 per cent was imposed during the period from October to May, without appreciable effect upon the contour of the pavement. The remaining 40 per cent during the season of high temperatures served virtually to destroy several of the weaker sections.

DISPLACEMENT GREAT DURING SUMMER SEASON

Twenty-eight sheet asphalt and five asphaltic concrete mixtures were tested in the second series. The sheet asphalts were laid directly upon the smooth

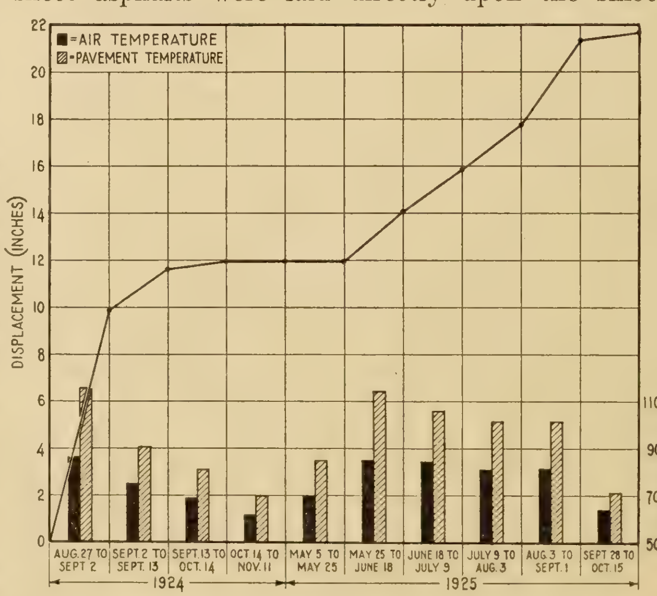


FIG. 2—Displacement and temperature of experimental asphaltic pavements

concrete foundation without the customary intermediate binder course. In this test, traffic was begun during the last few days of August and differences in the stability of the several mixtures were immediately developed. As the prevailing temperatures decreased the effect of traffic became less and less marked until, during the month preceding November 11, virtually

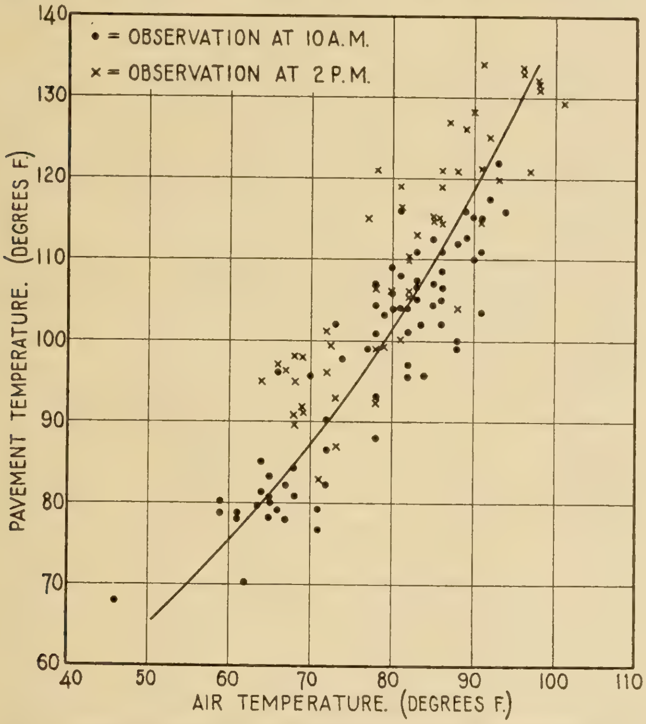


FIG. 3.—Relation between air and sheet asphalt pavement temperature. Observations taken one-half inch below the surface

no displacement was apparent. The average daily air temperature for the last period was 66°F. Traffic was suspended during the winter months, but was resumed early in the following May and continued until October 15. During this interval a complete cycle of temperature influences was observed. Stability of all mixtures at the low spring temperatures, shoving

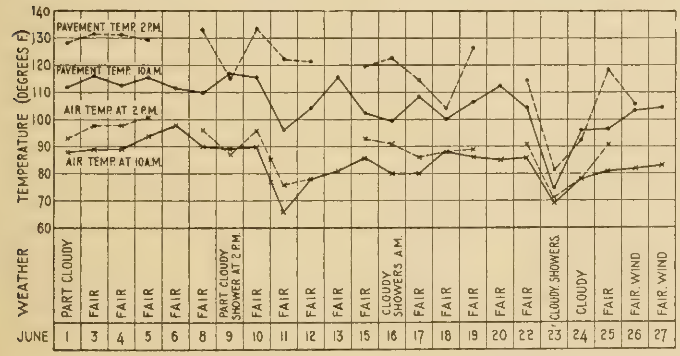


FIG. 4.—Record of corresponding temperatures of air and sheet asphalt pavement one-half inch below the surface

and rutting of weaker mixtures during the summer months, and once more an increased rigidity as cooler weather began to prevail.

Figure 2 indicates the average movement per 1,000 passages of the truck of all 33 sections of the second series with respect to the average daily air and pavement temperatures for the period or "traffic interval" between which measurements of movement were taken. The movement of each section is determined by averaging the total forward shove of two lines of 25 screws each across the 13-foot roadway. The points plotted on the curve are the averages of the 33 sectional movements. The high degree of plasticity of the pavements during the initial traffic interval ending September 2 may be significant. This period came not only in extremely hot weather, but followed immediately after construction, and it is probable that the apparent instability may be ascribed in considerable degree to the fact that the sections had not attained the ultimate compression to which they were susceptible under traffic. In substantiation of this theory, attention is directed to the fact that virtually equivalent temperatures during the following year resulted in far less displacement. Furthermore, in a considerable number of the mixtures at least 50 per cent of the total movement recorded during the entire test occurred in this short initial period.

By the latter part of the summer of 1925, five sections which carried excessive amounts of bitumen had deformed very badly. In the traveled areas their original internal structure was entirely disrupted and their resistance to displacement at high temperatures was

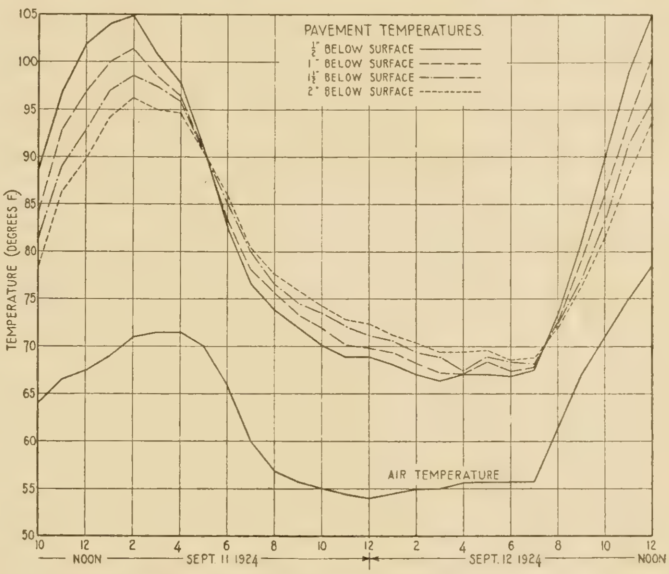


FIG. 5—Hourly record of internal temperature of sheet asphalt pavement

virtually destroyed. The condition of these pavements is reflected in the rather sharp rise in the curve of Figure 2 for the period beginning August 3.

It should be recognized that this chart does not pretend to measure the susceptibility of asphaltic pavements as a class to movement at the indicated temperatures. In these tests many mixtures remained stable at all temperatures whereas others, several of which were obviously of poor design, deformed very badly. Thus the plotted movements are representative only of a group of arbitrarily chosen mixtures.

PAVEMENT TEMPERATURES HIGHER THAN AIR TEMPERATURES

When exposed to direct sunlight the internal temperature of asphaltic pavements is probably always higher than that of the surrounding air. This difference is very likely slight during the winter months, but under summer conditions the bituminous mixture accumulates heat rapidly and its temperature rises faster than that of the air during the period of the day when it is subjected to direct action of the sun's rays. In these tests internal pavement temperatures 25 to 35° F. higher than the air temperatures were of common occurrence. The highest pavement temperature recorded was 140° F. and this was reached only a very few times. Figure 3 shows the relation between air and pavement temperatures, the latter obtained from the thermocouples placed one-half inch beneath the surface. The 10 o'clock and 2 o'clock readings are recorded separately. All plotted points represent data obtained on clear days, although winds of varying intensity and direction often prevailed. It will be noted that, in general, the same air temperature resulted in higher afternoon pavement temperatures indicating that gains by absorption exceeded losses through conduction and radiation.

Rain, clouds, or local shade result in an immediate decrease in the temperature of asphaltic pavements. The effect of the last-named factor was particularly noticeable in several of the mixtures which, during the afternoons of the late summer months, were shaded by a high bank surmounted by trees just west of the test pavement. As the line of shade crossed the location of the embedded thermocouples their temperatures abruptly decreased. In Figure 4 are plotted morning and afternoon observations of air and pavement temperatures, the latter being taken by means of the thermocouple placed one-half inch below the surface of the sheet asphalt mixture. The record extends over a period of approximately four weeks and illustrates characteristic reactions of pavement temperature to changes in climatic conditions.

There appears to be no extreme difference between the temperatures of asphaltic pavements at various depths beneath the surface of the customary 2-inch layer. While the pavement is subjected to direct sunlight the top was found always to be warmest. In these tests, however, differences in temperature at the one-half-inch depth and at the 2-inch depth never exceeded 15° F. On cloudy days the pavement temperatures were virtually uniform throughout, while rain, furnishing a medium for rapid losses of heat from the surface, frequently resulted in slightly higher temperatures at the bottom. This latter condition seems also to prevail at night. Figure 5 shows a series of 26 consecutive hourly temperature measurements taken at four depths in a sheet asphalt pavement.

Of particular interest in this figure are the uniformity of the temperature throughout the pavement at 5 p. m. and 7 a. m., the warmer condition of the protected under layers during the hours of darkness and the quick reversal to normal daytime conditions as direct sunlight once more strikes the pavement the following morning.

In common with all other structures, asphaltic pavements may be so designed that they will prove successful under some conditions but fail under others. Obviously, under no traffic, all 60 of the test sections involved in these experiments would have remained stable regardless of climatic conditions. Test data also show this practically to be the case when traffic was imposed during periods of average air temperature below 70° F. Higher temperatures aided traffic in developing instability in certain mixtures, but with lighter traffic it is certain that several of these would have been classed with those which proved satisfactory. It is clear, therefore, that pavement behavior, frequently explained on the basis of traffic only, should be analyzed also with regard to the prevailing climatic conditions at the time such traffic is imposed.

CONCRETE TESTS TO BE MADE IN NEW JERSEY

The New Jersey State Highway Department, in cooperation with the Bureau of Public Roads, has begun a series of concrete tests for the purpose of studying the relative concrete making properties of crushed stone and gravel used in concrete road construction in that State. The tests are being made in the State Highway Laboratory at Trenton, and involve the fabrication and testing of about 250 concrete beams 8 by 8 by 48 inches in size, as well as a large number of cylinders for compression tests.

The program calls for three series of tests. In the first series the workability of the concrete is to be kept constant, as nearly as possible, by means of the flow test, and the relative yield and strength of the concrete determined for each of several gradations both of crushed stone and gravel, using concrete proportions as given in the current New Jersey Standard Specifications. The object of this series is to determine the relative strength and yield of gravel concrete as compared with crushed stone concrete for several sizes and gradations of coarse aggregate.

In the second series an effort will be made to design concrete of a given strength by means of the water-cement ratio theory, for each type and gradation of coarse aggregate. The procedure to be followed in this series is essentially as follows. To each gradation and type of coarse aggregate, fine aggregate will be added in the following ratios by volume, 33 to 67, 36 to 64 and 40 to 60. To each of the above combinations water and cement in fixed ratio, depending on the strength desired, will be added until the desired workability has been reached. The end point in each case will be determined by means of the flow test, supplemented by the judgment of experienced concrete operators. Concrete specimens will then be made up in the proportions as determined by the trial method referred to, and the comparative strength, which should be constant, the comparative yield, and the comparative absorption will be determined.

In the third series of tests, specimens will be made in which the concrete mixture has been designed in accordance with the fineness modulus theory as given in "The Design and Control of Concrete Mixtures," a publication recently issued by the Portland Cement Association. The results obtained from this series will be used as a check against the results obtained in the second series.

Assuming a constant strength and a constant degree of workability, it is hoped to determine by means of these tests what grading of coarse aggregate and what proportions of fine to coarse will give the greatest yield of concrete for both crushed stone and gravel.

MOTOR VEHICLE REGISTRATIONS, REVENUE, AND GASOLINE TAXES FOR THE YEAR 1925

For 1925 the motor vehicle statistics have been amplified somewhat and are published in two tables; the table on page 50 showing the number and classes of vehicles registered and the table on page 51 showing the corresponding receipts.

REGISTRATION STATISTICS

This table shows by States the motor vehicle registrations, the tax-exempt cars, dealers' licenses issued, and operators' and chauffeurs' permits.

As far as possible all reregistrations are eliminated and to avoid duplication of the recorded cars of other States all nonresident registrations are excluded. The grand total of registered motor cars and trucks has been divided into two classes based on their use, either as passenger-carrying, or commodity-carrying vehicles, the latter including tractors and motor-driven road equipment.

In the first column is the grand total of all registered and taxed motor cars and trucks, which is the total of columns 2 and 3. Column 2 shows the passenger cars, including all privately owned passenger automobiles, taxis and passenger cars for hire, and busses. A few States (noted in column 3) have included busses with motor trucks, but nearly all busses are included in column 2. Column 3 shows the commodity or freight carrying cars, which include all motor trucks, road tractors, and all motorized road vehicles, not primarily used for the transportation of passengers. Tractors used for farm purposes have been excluded although these are registered and pay a fee in a few States. Only 12 States record road tractors separately, amounting in 1925 to 2,749 tractors. The other States include road tractors with trucks.

Considerable interest has been shown in the number of taxis and cars for hire and busses. In the absence of sufficiently complete statistics from registration offices, data have been taken from other sources and shown under the heading "Special list of passenger cars for hire." The list in column 4 taken from Internal Revenue Bureau records shows the number of taxis and passenger cars for hire having seating capacity of two to seven persons for which an occupational Federal tax of $10 was paid by the owners during the fiscal year ending June 30, 1925. A few States supplied this data as of December 31, and where available, it has been used. The list of busses shown in column 5 is taken from the February issue of "Bus Transportation," and is only a partial list. This data has not been used in connection with the official figures summarized in column 2, except in a few States, as noted, which record passenger cars for hire and busses separately.

Registered trailers and motor cycles for which fees are paid are shown. The trailers in column 6 cover semitrailers (two wheels) and trailers (four wheels). In general, trailers are hauled by road tractors and carry commodities or freight. There are some two-wheel trailers used with passenger cars for "camp kits" but these are not segregated in the records. Motor cycles are listed in column 7 and include cycles with or without side cars.

Tax-exempt official cars are shown for all States which record these cars and trucks. The total number of United States cars and trucks given was obtained from the United States Budget Bureau. Under this heading is also shown official motor cycles, which are recorded in less than half the States.

The next to the last column shows the 1924 grand total with 1,696 tractors added to formerly published figures (in States noted) in order to be comparable with the 1925 grand total registered motor cars and trucks.

MOTOR VEHICLE REGISTRATION RECEIPTS AND DISPOSAL OF FUNDS

The first column of this table on page 51 shows the total gross receipts. The next eight columns show various items which make up the gross receipts as segregated by 33 States and the District of Columbia. The States starred in the column of States are the ones which reported the complete details, and these States have been summarized to make a subtotal called "Detailed total." The details of registration receipts correspond with the number of vehicles in the table on page 50 wherever reported. Under "Miscellaneous receipts" are shown dealer's license fees, chauffeurs' and operators' permits and other miscellaneous receipts, the latter covering many items such as reregistration fees, nonresident registration fees, traffic fines (if these are included in the motor vehicle fund) certificates of title, duplicate tags, etc.

The collection and administration expenses of the motor vehicle license offices are generally deducted before final division is made allocating certain shares to the State highways, to county or other local rural roads to pay retirement and interest on State road and bridge bonds, and for other purposes, as noted. Several States as noted pay collection and administration expenses out of State appropriations. In many cases a stated lump sum is authorized for this expense to be deducted from motor vehicle fund, or else a certain percentage of gross receipts is allowed.

GASOLINE TAXES

Data on gasoline tax rates and collections are shown on page 52. This table shows the gross receipts and the net receipts after the deduction of refunds allowed by law. Some States have no provision for refunds and in such cases gasoline used for purposes other than motor vehicles is included. Collection costs are shown where paid out of gasoline tax earnings.

Motor-vehicle registrations for the year 1925

State	Registered motor vehicles, individually and commercially owned: Grand total registered motor cars and trucks, 1925	Registered motor vehicles, individually and commercially owned: Passenger automobiles, taxis, and busses	Registered motor vehicles, individually and commercially owned: Motor trucks and road tractors	Special list of passenger cars for hire [1]: Taxis, etc.	Special list of passenger cars for hire [1]: Busses	Other registered vehicles: Trailers	Other registered vehicles: Motorcycles	Tax-exempt official vehicles and motor cycles [2]: United States cars	Tax-exempt official vehicles and motor cycles [2]: State and local cars	Tax-exempt official vehicles and motor cycles [2]: Motorcycles (official)	Number of licenses, or permits (autos.): Dealers [4]	Number of licenses, or permits (autos.): Operators	Number of licenses, or permits (autos.): Chauffeurs	Grand total registered motor cars and trucks, 1924 [3]	Per cent increase 1925 over 1924
Alabama	194, 580	171, 387	23, 193	2, 710	778	480	524				2, 599		2, 105	157, 262	23. 7
Arizona	68, 029	59, 798	8, 231	[5] 568	569		310		759	49	330		339	57, 828	17. 6
Arkansas	183, 589	159, 511	24, 078	[5] 2, 398	378	918	263		572	30				141, 983	29. 3
California	1, 440, 541	1, 225. 796	214, 745	5, 210	4, 017	27, 542	11, 177		[6] 18,647	583	11, 977		106, 230	1, 319, 394	9. 2
Colorado	240, 097	221, 513	18, 584	2, 416	816	82	1, 862				3, 206	20, 079	7, 776	213, 247	12. 6
Connecticut	250, 669	213, 486	37, 183	[5] 2, 301	904	332	3, 886		3, 139	148	4, 386			[7] 217, 236	15. 4
Delaware	40, 140	32, 550	7, 590	192	138	166	375				763	40, 841	3, 555	35, 136	14. 2
Florida	286, 388	237, 435	49, 953	[5] 3, 356	1, 253	1, 062	1, 200				2, 016		5, 656	195, 128	46. 8
Georgia	248, 093	217, 578	30, 515	1, 969	715		994				727		2, 921	207, 688	19. 4
Idaho	81, 506	73, 896	7, 610	558	570	168	518		1, 050		324		448	69, 227	17. 7
Illinois	1, 263, 177	1, 101, 943	[8] 161, 234	10, 374	3, 289	3, 777	6, 603		None.		2, 000		100, 000	1, 119, 236	12. 8
Indiana	725, 410	630, 554	94, 856	3, 648	1, 896	5, 068	4, 525		525		2, 242		40, 247	651, 705	11. 3
Iowa	659, 202	613, 412	45, 790	2, 284	1, 321	125	2, 303		2, 500	90				616, 128	7. 0
Kansas	457, 033	409, 968	[8] 47, 065	2, 140	696		1, 434		2, 014					410, 891	11. 2
Kentucky	261, 647	235, 020	[8] 26, 627	2, 981	1, 120		703		1, 169		1, 119		8, 867	229, 804	13. 8
Louisiana	207, 000	176, 000	31, 000	1, 477	672		520						10, 000	178, 000	16. 3
Maine	140, 499	116, 229	24, 270	[5] 2, 716	376	790	1, 293		882	88	1, 070	162, 435	6, 150	[7] 127, 598	10. 1
Maryland	234, 247	222, 173	12, 074	[5] 3, 477	636	586	4, 619					109, 747	38, 185	[7] 198, 465	18. 0
Massachusetts	646, 153	554, 813	91, 340	6, 254	1, 857	702	9, 401		800	400	2, 011	698, 378	([9])	570, 578	13. 2
Michigan	989, 010	885, 524	[8] 103, 486	3, 325	2, 161	10, 592	3, 387		3, 353		1, 958	197, 547	75, 621	867, 545	14. 0
Minnesota	569, 694	524, 879	44, 815	1, 833	932	1, 912	2, 923			21	1, 943			503, 437	13. 1
Mississippi	177, 262	159, 134	18, 128	1, 555	2, 049		100							134, 680	33. 6
Missouri	604, 166	543, 426	60, 740	4, 821	1, 407	1, 087	1, 984		1, 317	5				540, 500	11. 8
Montana	94, 656	82, 135	12, 521	583	323		252		1, 000					79, 695	18. 8
Nebraska	338, 719	301, 716	[8] 37, 003	1, 256	323	807	1, 207							308, 715	9. 7
Nevada	21, 169	18, 069	3, 100	137	175	20	120		393	13				18, 118	16. 8
New Hampshire	81, 498	72, 472	9, 026	1, 903	635	497	1, 701		300		507	60, 772	31, 903	[7] 71, 149	14. 5
New Jersey	580, 554	469, 156	111, 398	5, 367	2, 401	1, 389	7, 730		4, 469	771				[7] 504, 470	15. 0
New Mexico	49, 111	47, 470	1, 641	391	308	88	209			3				41, 680	17. 8
New York	1, 625, 583	1, 346, 665	278, 918	26, 079	3, 966	5, 051	18, 642		10, 588	1, 192	4, 703			1, 412, 879	15. 0
North Carolina [10]	340, 287	311, 384	28, 903	2, 102	2, 446	500	863		4, 110					302, 232	12. 6
North Dakota	144, 972	133, 791	[8] 11, 181	375	1, 078		443							117, 346	23. 5
Ohio	1, 346, 400	1, 179, 400	167, 000	5, 354	4, 103	9, 000	12, 650		4, 200					1, 241, 600	8. 4
Oklahoma	424, 345	393, 047	31, 298	2, 212	1, 231		817							369, 903	14. 7
Oregon	216, 553	199, 517	17, 036	732	698	([11])	2, 547				598	51, 084	15, 188	192, 615	12. 4
Pennsylvania	1, 330, 433	1, 149, 074	181, 359	6, 937	2, 615	2, 821	15, 234		9, 750	888	24, 105	1, 570, 219	([9])	[7] 1, 228, 845	8. 3
Rhode Island	101, 756	84, 337	17, 419	1, 431	[12] 262	59	1, 343		458	64		117, 252	([9])	95, 482	6. 6
South Carolina	168, 496	153, 343	15, 153	1, 646	473	824	173		1, 261	51				161, 753	4. 2
South Dakota	168, 028	154, 141	[8] 13, 887	452	205		345		763		959			142, 396	18. 0
Tennessee	244, 626	221, 712	22, 914	2, 301	720		627			50	516			204, 680	19. 5
Texas	975, 083	886, 362	88, 721	6, 454	1, 260	4, 600	2, 228		1, 302	551				[7] 801, 833	21. 6
Utah	90, 500	79, 170	11, 330	312	460	200	719		1, 015					68, 316	32. 5
Vermont	69, 576	64, 566	5, 010	1, 134	153		718							61, 179	13. 7
Virginia	282, 650	246, 950	35, 700	2, 543	1, 373	440	1, 590		2, 435	125				261, 945	7. 9
Washington	328, 442	281, 452	46, 990	[5] 1, 633	1, 574	1, 595	2, 879			141				295, 443	11. 1
West Virginia	217, 589	190, 257	27, 332	1, 569	856	345	1, 432				7, 700	64, 702	26, 648	[7] 191, 085	13. 9
Wisconsin	594, 386	528, 090	[8] 66, 296	2, 535	820	([11])	3, 443		555	80	2, 700			525, 221	13. 2
Wyoming	47, 711	42, 547	5, 164	347	553		220		203					43, 639	9. 3
District of Columbia	103, 092	89, 790	13, 302	1, 182	265	No fee.	1, 312					17, 503	3, 126	88, 762	16. 1
Totals	19, 954, 347	17, 512, 638	2, 441, 709	145, 530	[1] 57,826	83, 625	140, 348	[13] 17, 400	79, 529	5, 343				[7] 17, 593, 677	13. 4

[1] Special list from sources other than registration offices. Taxis and cars for hire taken largely from Internal Revenue Bureau report. List of busses taken largely from "Bus Transportation." These vehicles are included in the grand total given in the first column.
[2] Not included in the grand total given in the first column.
[3] Total formerly published has been revised to include 1696 road tractors to give a figure comparable to the 1925 total which for the first time includes road tractors.
[4] As reported by States and is not complete.
[5] As reported by motor vehicle bureau.
[6] Includes 7,728 Public Service Corporation vehicles which are tax exempt.
[7] Includes road tractors formerly not included.
[8] Includes busses as reported by State.
[9] Included with operators licenses.
[10] Only data from July 1 to Dec. 31 reported.
[11] Included with motor trucks and tractors.
[12] Total reported by State which is larger than that shown in "Bus Transportation."
[13] From records of the Bureau of the Budget. War Department vehicles not included.

Receipts from motor vehicle registration fees, etc., for the year 1925

States	Total gross receipts [1]	Subdivision of registration receipts [2]					Miscellaneous receipts				Disposition of gross receipts			
		Motor cars			Other vehicles						For highway purposes			
		Total motor cars	Passenger cars and busses	Trucks and tractors	Trailers	Motor cycles	Dealers' licenses	Chauffeur and operator permits	Miscellaneous	Collection and administration	State highways	Local roads	State road bonds	For other purposes
Alabama	$2,511,129	[3] $2,494,820					$2,599	$10,410	$3,299	[4] $105,527	$769,874	$486,490	$1,138,828	[5] $10,410
Arizona	405,592	385,032				$644	3,649	1,695	14,573	18,000	[6] 387,592			
Arkansas	3,150,000	(7)								12,000	1,731,000	583,000	824,000	
California*	7,816,298	6,754,002	$4,081,130	$2,672,872	$209,185	39,956	42,251	258,684	512,220	951,076	3,432,611	3,073,607		[8] 359,004
Colorado*	1,430,299	1,336,392	1,127,149	209,243	1,140	3,724			89,043	71,515	679,392	679,392		
Connecticut*	5,644,247	4,303,483	3,178,878	1,124,605	7,853	15,376			1,317,535		5,644,247			
Delaware*	680,700	517,004	378,265	138,739	2,269	1,485	7,990	133,720	18,232		680,700			
Florida*	3,645,628	3,449,052	2,536,383	912,669	18,927	4,803	24,435	9,019	139,392	261,220	2,538,306	846,102		
Georgia*	3,010,415	2,952,609	2,473,485	479,124		4,081	42,700	5,594	5,431	98,297	2,912,118			
Idaho*	1,192,587	1,155,174	967,860	187,314	3,711	2,450	19,515	896	10,841	(9)	140,444	1,037,226	14,917	
Illinois*	12,969,754	12,111,679	9,259,929	2,851,750	46,004	23,963	88,050	355,519	344,539	(10)	9,982,450		2,987,304	
Indiana*	4,649,663	4,318,734	3,300,396	1,018,338	17,352	8,336	53,950	74,567	176,724	205,681	4,443,982			
Iowa	9,741,103	(7)								713,036	5,758,141		3,030,325	[11] 239,601
Kansas	4,610,090	(7)								230,505	3,284,689	1,094,896		
Kentucky*	3,780,062	3,664,979	2,864,448	800,531		5,581	31,012	16,819	61,671	132,105	3,247,733	400,224		
Louisiana	3,400,045	3,343,049				2,600		54,396		40,000	3,360,045			
Maine*	2,182,135	1,671,096	1,330,814	340,282	2,615	7,936	324,870	32,952	142,666	[12] 254,526	1,302,196		552,647	[13] 72,766
Maryland*	2,576,301	2,006,322	1,744,423	261,899	11,978	15,682		262,595	279,724	[14] 250,000	2,326,301			
Massachusetts*	9,843,901	7,346,952	5,794,224	1,552,728	14,795	47,069	59,700	1,396,756	978,629	921,514	8,922,387			
Michigan*	14,526,002	13,107,863	10,160,579	2,947,284	121,435	13,234	86,563	241,782	955,125	300,000	7,356,467	6,000,000		[15] 869,535
Minnesota*	9,744,834	9,651,795	8,654,290	997,505	6,847	11,743	34,092		40,357	(16)	6,294,834		3,450,000	
Mississippi*	1,530,000	1,529,150	1,377,000	152,150		850				45,900		1,484,100		
Missouri	7,267,098	(7)								432,023	6,835,075			
Montana*	915,253	914,878	788,125	126,753		375				32,000		883,253		
Nebraska*	3,936,458	3,791,628	3,141,477	650,151	3,456	4,902			136,472	98,411	1,151,414	2,686,633		
Nevada	209,197	208,401				600			196	10,584	114,225	3,138	81,250	
New Hampshire	1,736,094	1,383,969			(17)	9,556	28,401	229,535	84,633	114,610	1,613,804			[18] 7,680
New Jersey*	10,515,323	7,582,255	4,527,893	3,054,362	45,895	15,460	63,661	1,983,948	824,104	1,177,057	5,552,266	3,725,000		[19] 61,000
New Mexico*	457,874	447,001	403,344	43,657	570	728			9,575	31,991	283,922	141,961		
New York*	25,506,245	22,502,688	15,675,072	6,827,616	36,168	85,186	153,745		2,728,458	[20] 372,848	18,876,461	6,241,060		[18] 15,876
North Carolina	[21] 8,359,844	(7)								149,761	8,210,083		(22)	
North Dakota*	1,083,573	1,049,324	935,031	114,293		1,397			32,852	150,000	401,787	401,786	130,000	
Ohio	13,147,231	(7)								(23)	6,573,616	6,573,615		
Oklahoma	4,576,572	(7)								(24)	187,858	3,978,022		[25] 410,692
Oregon*	5,370,202	5,207,691	4,440,577	767,114	(17)	14,629	17,570	77,107	53,205	200,000		1,292,551	3,877,651	
Pennsylvania*	21,926,972	16,934,504	11,568,692	5,365,812	29,277	41,932	296,887	1,721,187	2,903,185	[26] 2,563,137	18,952,448			[27] 411,387
Rhode Island*	1,863,955	1,432,561	1,059,054	373,507	1,003	5,009	13,340	234,504	177,538	306,492	1,557,463			
South Carolina*	2,366,076	2,106,271	1,784,735	321,536	13,710	1,567	25,670	1,141	217,717	187,729	1,736,716			[28] 441,631
South Dakota*	2,445,112	2,403,501	2,143,944	259,557		1,630	23,975		16,006	21,511	1,222,556	1,201,045		
Tennessee	3,060,948	(7)								54,243	3,006,705			
Texas	13,477,931	8,976,151				11,140			4,490,640	476,146	9,368,187	3,633,598		
Utah	554,235	(7)											[29] 554,235	
Vermont*	1,497,146	1,265,611	1,145,126	120,485		5,000			226,535	82,037	1,415,109			
Virginia*	4,300,950	3,947,402	3,414,997	532,405	4,594	7,576			341,378	(30)	4,122,018			[31] 178,932
Washington*	4,980,026	4,848,572	3,774,828	1,073,744	32,715	15,414			83,325	240,059	4,665,195	74,772		
West Virginia*	3,354,247	3,022,617	2,470,524	552,093	2,577	5,902	40,910	137,620	144,621	264,386	783,573		2,000,000	[32] 306,288
Wisconsin*	7,896,210	7,659,722	6,309,848	1,349,874	(17)	21,140	86,775		128,573	380,000	5,626,210	1,875,000	(33)	[19] 15,000
Wyoming*	482,857	470,459	378,169	92,290		1,054			11,344	(10)			482,857	
District of Columbia*	291,207	111,758	98,456	13,302		1,312		49,809	128,328	[34] 36,820	[35] 254,387			
Detailed total [2]	184,412,512	161,574,729	123,289,145	38,285,584	634,076	436,482	1,537,661	6,994,219	13,235,345					
Grand total	260,619,621									11,992,747	177,706,587	48,396,471	19,124,014	3,399,802

[1] Total funds derived from operation of motor-vehicle laws, including registration fees, licenses, permits, fines, etc.
[2] Only the 33 States and the District of Columbia reported data in full detail, 6 gave partial details and 9 gave no details, therefore the detailed total is less than that of the first column.
[3] Includes all registered vehicles.
[4] Includes $62,370 for probate judges.
[5] Amount from licenses of taxi chauffeurs allotted to State general fund.
[6] For maintenance work.
[7] No details given.
[8] Traffic officers' expenses, deducted from county's share of net receipts.
[9] Special State appropriation through State highway fund.
[10] Special State appropriation.
[11] For State highway commission maintenance.
[12] Includes $153,531 for motor-vehicle law enforcement.
[13] Expenses of State highway commission.
[14] Estimated.
[15] Expenses of motor-vehicle theft department.
[16] Estimated at $302,600 paid from State appropriation
[17] Included under motor cars.
[18] Refunds.
[19] Toll bridge commission.
[20] Collection fees of county clerks in addition to the expenses of 7 city offices, $1,857,900, taken from general State fund.
[21] For period of 6 months, July 1 to December 31, as registration year begins July 1.
[22] Interest and sinking fund requirements included in State highway amount.
[23] Special legislative appropriation of $363,659.
[24] Expenses from State highway department fund.
[25] State general fund to July 1, 1925; not to receive any share after this date.
[26] $1,420,048 expended for administration and balance for administration of road work by State highway department.
[27] For State highway patrol.
[28] Includes $374,140 refund by amendment to law and $67,491 to State genera fund.
[29] Includes amount spent on collection and administration.
[30] State appropriation of $296,969.05.
[31] Operation of auto theft law.
[32] State road commission expenses.
[33] Bond payments included with other items.
[34] All money collected deposited in U. S. Treasury. This amount is the appropriation for expenses of administration.
[35] Amount to balance with gross receipts. The United States appropriations for streets is much higher.

Gasoline taxes for the year 1925

States	Gross tax assessed, prior to deduction of refunds	Exemption refunds: (Deduct from gross tax)	Total tax earnings on fuel for motor vehicles	Disposition of total tax earnings				Tax rates, 1925			Net gallons of gasoline taxed and used by motor vehicles	Estimated additional gallons (not taxed) used by motor vehicles
				Collection costs	Construction and maintenance of rural roads		For other purposes	Cents per gallon		Date of rate change		
					State highways	Local roads		Jan. 1	Dec. 31			
Alabama	$2,140,802		$2,140,802	$9,461		[1] $2,131,341		2	2		107,040,092	
Arizona	1,035,551	$179,600	855,951		$427,976	427,975		3	3		28,531,686	
Arkansas [2]	3,230,559	280,199	2,950,360		1,357,360	[3] 1,593,000		4	4		73,759,002	
California	16,150,387	1,193,598	14,956,789	7,393	7,229,248	7,229,248	[4] $490,900	2	2		747,839,462	
Colorado	1,991,531	30,585	1,960,946		980,473	980,473		2	2		97,377,858	
Connecticut	1,908,809		1,908,809		1,908,809			1	2	July 1	122,230,292	
Delaware	350,580	8,499	342,081		342,081			2	2		17,104,050	
Florida	7,657,507		7,657,507	6,000	5,549,978	2,101,529		3	4	June 6	210,323,517	
Georgia	4,418,824		4,418,824	4,200	1,641,248	1,386,688	[5] 1,386,688	3	3½	Aug. 26	138,802,152	
Idaho	932,064	36,621	895,443	9,466	885,977			2	3	Mar. 1	30,809,320	
Illinois	None.							0	0	No tax.		530,534,340
Indiana	7,832,462	179,413	7,653,049	12,436	5,200,637	2,439,976		2	3	Apr. 1	272,980,870	
Iowa	3,568,184	63,069	3,505,115	5,520	1,151,144	2,302,289	[6] 46,162	0	2	Apr. 16	175,255,740	51,796,350
Kansas	3,000,253	95,059	2,905,194		2,905,194			0	2	May 1	145,259,690	52,911,710
Kentucky	3,041,560		3,041,560		3,041,560			3	[7] 3		101,385,318	
Louisiana	2,339,543		2,339,543		[8] 2,339,543			2	2		116,939,139	
Maine	1,283,874	15,526	1,268,348	5,596	[9] 1,262,752			1	3	July 11	56,513,741	
Maryland	2,022,986	45,950	1,977,036	2,500	[10] 1,579,629		[11] 394,907	2	2		98,851,813	
Massachusetts	None.							0	0	No tax.		274,615,025
Michigan	8,742,392	506,314	8,236,078	41,358	[12] 6,694,720	[13] 1,500,000		0	2	Feb. 1	411,803,894	19,856,370
Minnesota	3,989,282	125,342	3,863,940		3,863,940			0	2	May 1	199,464,097	57,908,930
Mississippi	2,494,274		2,494,274	1,800	[8] 1,224,976	[8] 1,203,715	[14] 63,783	3	3		83,142,469	
Missouri	4,234,070	74,955	4,159,115	23,429	4,135,686			2	2	Jan. 1	207,955,474	
Montana	674,710		674,710		101,207	371,090	[15] 202,413	2	2		33,735,497	
Nebraska	2,202,236	8,434	2,193,802	4,963	2,188,839			0	2	Apr. 1	109,690,122	26,570,900
Nevada	335,446	16,741	318,705		159,353	159,352		2	4	do	8,850,407	
New Hampshire	716,140	9,068	707,072		707,072			2	2		35,353,585	
New Jersey	None.							0	0	No tax.		249,638,220
New Mexico	537,356		537,356	26,868	[8] 510,488			1	3	Mar. 17	20,490,892	
New York	None.							0	0	No tax.		715,256,520
North Carolina	6,238,508	156,130	6,082,378		6,082,378			3	4	Feb. 21	161,371,522	
North Dakota	649,416	15,000	634,416		224,095		[15] 410,321	1	1		64,941,557	
Ohio	9,133,785	123,835	9,009,950		[8] 4,054,478	[8] 2,252,487	[16] 2,702,985	0	2	Apr. 18	450,497,522	99,560,500
Oklahoma	5,143,517		5,143,517		3,351,898	1,791,619		2½	3	Mar. 23	176,753,177	
Oregon	3,065,151	156,056	2,909,095	6,553	2,902,542			3	3		96,969,835	
Pennsylvania	8,352,798		[17] 8,352,798		3,136,819	2,105,917	[15] 3,110,062	2	2		414,096,490	
Rhode Island	318,357		318,357		318,357			0	1	Apr. 29	31,835,668	3,576,640
South Carolina	3,870,588	5,185	3,865,403		2,186,152	1,512,889	[18] 166,362	3	5	Mar. 23	83,962,562	
South Dakota	2,122,406	274,808	1,847,598		1,847,598			2	3	Mar. 10	64,024,928	
Tennessee	3,407,886		3,407,886	22,768	3,385,118			2	3	Feb. 9	122,000,680	
Texas	4,641,784		4,641,784		3,481,338		[19] 1,160,446	1	1		464,178,427	
Utah	1,064,004		1,064,004	3,750	[20] 1,060,254			2½	3½	Apr. 1	32,217,216	
Vermont	502,272		502,272		[8] 502,272			1	2	Feb. 26	25,863,167	
Virginia	3,863,117	161,166	3,701,951	5,604	2,464,231	1,232,116		3	[21] 3		123,398,365	
Washington	3,205,114	184,302	3,020,812		3,020,812			2	2		151,040,586	
West Virginia	2,222,329	35,590	2,186,739	7,500	[22] 2,179,239			2	3½	July 1	76,331,660	
Wisconsin	4,155,469	123,793	4,031,676	10,000	4,021,676			0	2	Apr. 1	201,583,789	48,830,860
Wyoming	460,972	4,675	456,297	228	456,069			1	2½	do	20,746,056	
District of Columbia	896,568	6,970	889,598				[23] 889,598	2	2		44,479,898	
Total			146,028,940	217,393	102,065,216	32,721,704	11,024,627	Av. 2.26			6,457,783,284	2,131,056,365

[1] For maintenance only.
[2] In addition $438,436 collected as motor oil tax at a rate of 10 cents per gallon.
[3] Includes $873,240 payments on county road and bridge bonds.
[4] Delinquent taxes uncollected not disposable in 1925.
[5] To State treasury.
[6] Unaccounted for; probably delinquent taxes.
[7] Tax increased to 5 cents effective February 21, 1926.
[8] For maintenance only.
[9] Includes $282,913 for maintenance.
[10] For maintenance and reconstruction.
[11] For maintenance of Baltimore streets.
[12] Includes $3,000,000 for interest and retirement payments on State road bonds.
[13] Payments to counties on State award highways.
[14] For sea-wall in Harrison County.
[15] For State general fund.
[16] Maintenance of municipal streets.
[17] Includes $70,868 paid-in delinquent taxes of former years.
[18] Covers part of first four months of year only, as new law excludes State general fund from share in gasoline tax fund.
[19] For free school fund.
[20] Includes $460,000 payment of interest and to sinking fund on State road bonds.
[21] Tax increased to 4½ cents effective Mar. 11, 1926.
[22] Includes $1,520,463 payment of interest on State road bonds.
[23] For improvement and repair of Washington streets.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
390. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
*532. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660. Highway Cost Keeping. 10c.
670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691. Typical Specifications for Bituminous Road Materials. 10c.
*724. Drainage Methods and Foundations for County Roads. 20c.
*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
1216. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1259. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULARS

No. 94. TNT as a Blasting Explosive.
331. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

No. 60. Federal Legislation Providing for Federal Aid in Highway Construction.

FARMERS' BULLETINS

No. *338. Macadam Roads. 5c.
*505. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *727. Design of Public Roads. 5c.
*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.
914. Highways and Highway Transportation.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

*Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

MARCH 31, 1926

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2,863,197.86	611.8	$ 10,735,200.15	$ 5,141,722.18	588.9	$ 6,008,211.95	$ 2,854,587.23	287.8	$ 133,112.16	$ 66,556.08		$ 3,423,391.65	Alabama
Arizona	9,580,133.43	5,016,119.94	613.8	1,139,122.72	706,313.84	98.4	1,151,770.40	757,869.71	82.8	176,977.10	108,150.70	26.7	3,028,794.81	Arizona
Arkansas	13,310,190.08	5,380,181.73	1,048.9	4,278,673.15	1,904,193.17	222.9	5,310,982.30	2,555,646.53	345.6	536,670.32	236,160.00	37.4	1,529,622.57	Arkansas
California	22,346,175.99	10,713,249.61	894.8	3,136,696.93	1,471,494.55	112.7	12,290,177.59	6,035,504.13	351.3				3,846,566.71	California
Colorado	11,876,703.94	6,067,814.34	651.2	1,151,021.44	598,197.01	54.6	4,517,653.22	2,278,314.49	234.7	225,774.02	126,625.34	12.6	3,254,860.82	Colorado
Connecticut	4,558,639.29	1,819,368.66	101.6	750,965.91	250,212.14	13.4	1,901,818.16	590,259.68	29.1	582,417.39	144,362.95	9.1	1,529,477.57	Connecticut
Delaware	4,281,559.81	1,496,190.65	107.1	455,744.13	212,544.95	12.3	980,042.66	388,784.75	22.7	571,272.05	59,220.00	13.8	317,317.65	Delaware
Florida	2,959,273.72	1,405,487.97	96.3				10,346,486.92	4,918,554.60	295.4	130,171.16	65,085.58	13.7	1,695,825.85	Florida
Georgia	20,156,002.37	9,406,366.46	1,478.3	3,584,313.71	1,743,792.54	232.3	11,360,710.24	5,599,100.33	689.2	1,210,588.87	600,876.20	56.6	1,081,817.47	Georgia
Idaho	9,394,676.80	4,815,332.26	600.1	1,453,421.38	912,273.25	103.5	2,018,134.62	1,233,555.87	127.2	644,306.66	357,911.76	16.8	1,240,553.86	Idaho
Illinois	40,010,481.10	18,640,076.28	1,236.2	3,012,732.19	1,447,994.11	99.5	6,344,557.34	3,068,862.83	218.0				6,675,264.78	Illinois
Indiana	13,639,172.65	6,562,455.68	422.1	2,144,221.15	1,035,593.69	73.2	17,087,806.75	8,003,335.26	464.7	421,901.51	208,509.47	12.8	2,394,460.90	Indiana
Iowa	27,272,285.21	11,107,492.99	1,996.9	722,462.52	320,365.06	36.5	8,105,894.60	3,660,339.59	498.3	3,541,207.32	1,683,812.99	244.2	2,713,552.37	Iowa
Kansas	26,399,695.77	9,755,273.32	831.4	5,319,314.12	2,349,067.90	286.1	10,642,820.38	4,128,274.72	539.9	1,991,243.72	625,729.10	127.9	2,606,065.96	Kansas
Kentucky	14,832,324.28	6,205,994.59	584.9	3,521,723.17	1,381,242.75	96.4	7,187,922.44	3,276,846.30	308.8	257,788.45	111,997.22	10.2	2,236,728.14	Kentucky
Louisiana	11,939,424.97	5,279,870.86	927.6	1,622,969.92	735,731.50	103.4	3,420,155.43	1,659,669.90	160.0				1,597,135.74	Louisiana
Maine	8,174,281.31	3,907,870.33	281.4	573,271.45	284,637.06	22.2	1,809,495.54	706,814.24	57.5	92,205.65	46,102.82	4.7	1,519,403.55	Maine
Maryland	8,132,506.90	3,849,383.15	294.4	2,711,863.25	1,234,208.07	126.9	249,190.03	114,397.50	12.7				727,068.28	Maryland
Massachusetts	14,047,656.22	5,467,661.28	300.6	2,198,203.20	591,531.01	40.1	4,484,811.18	1,267,266.18	60.7	1,161,802.28	285,284.20	16.9	2,496,983.33	Massachusetts
Michigan	16,234,000.80	7,328,316.91	612.6	9,286,862.11	4,283,204.37	335.7	9,147,721.14	4,128,580.06	218.0	382,294.00	178,607.13	17.2	4,423,656.53	Michigan
Minnesota	30,415,685.89	12,738,642.04	2,721.2	5,271,152.99	2,284,974.52	397.1	7,803,579.46	2,495,300.00	522.2	1,288,037.91	314,000.00	71.7	1,758,863.44	Minnesota
Mississippi	10,292,285.79	4,988,702.73	803.4	3,000,939.78	1,499,508.16	221.4	8,026,600.06	4,007,044.36	419.8	585,436.85	283,918.53	36.0	1,348,844.22	Mississippi
Missouri	17,368,156.57	8,219,411.43	1,118.9	8,614,993.14	4,162,082.75	311.6	20,833,047.86	8,338,598.72	586.1	1,776,184.96	788,225.64	68.0	1,278,117.46	Missouri
Montana	10,156,600.41	5,317,523.15	921.6	1,244,383.40	1,015,942.74	133.3	1,380,058.51	930,089.13	107.3	945,723.12	513,821.09	110.2	5,647,508.89	Montana
Nebraska	9,306,374.36	4,389,523.50	1,570.6	1,521,442.87	736,793.25	120.3	11,107,158.19	5,502,447.60	1,081.1	2,065,372.88	1,030,828.86	206.2	2,976,641.79	Nebraska
Nevada	4,917,465.69	3,088,299.78	357.3	2,029,124.24	1,587,127.57	160.8	3,678,708.88	3,097,062.10	335.3				1,022,725.55	Nevada
New Hampshire	4,165,687.86	1,986,226.87	208.1	781,097.31	369,244.23	27.9	632,628.59	296,233.16	19.2	45,925.79	22,962.89	1.4	494,824.85	New Hampshire
New Jersey	11,961,357.45	3,820,679.99	219.1	2,683,712.79	726,976.44	47.4	7,872,904.26	2,948,900.47	44.1	420,870.40	66,450.00	4.4	904,413.10	New Jersey
New Mexico	8,717,999.18	4,914,070.61	1,081.3	3,545,064.81	2,313,941.49	329.6	1,483,968.37	978,752.14	116.4	74,555.37	46,757.90	5.4	2,718,863.86	New Mexico
New York	28,597,769.67	12,229,076.53	831.5	9,095,913.25	3,668,925.81	230.8	31,442,495.58	9,257,492.24	586.5	8,313,370.00	2,009,390.00	134.4	6,880,310.42	New York
North Carolina	21,014,450.41	8,746,454.59	1,119.8	3,923,647.63	1,599,863.21	94.0	8,456,432.34	3,619,780.38	204.2	870,202.76	435,101.38	14.9	1,316,006.44	North Carolina
North Dakota	10,829,263.82	5,268,930.47	1,917.5	1,420,744.22	731,277.64	275.6	3,482,106.39	1,723,110.51	426.5	1,393,357.25	723,263.22	162.7	2,302,077.16	North Dakota
Ohio	41,572,252.81	15,244,993.93	1,191.1	6,059,391.44	2,107,214.63	171.8	8,342,639.32	3,301,752.39	264.2	2,788,407.93	906,308.43	70.3	4,171,526.62	Ohio
Oklahoma	20,787,024.94	9,672,890.34	852.2	4,319,658.63	2,006,425.89	174.3	5,371,626.48	2,559,148.85	242.2	234,268.05	110,765.51	7.3	1,710,556.41	Oklahoma
Oregon	14,388,188.70	7,142,364.63	794.6	2,325,783.62	1,256,645.21	111.3	2,290,319.28	1,378,618.00	119.0	1,434,441.46	659,928.75	46.7	441,790.41	Oregon
Pennsylvania	43,054,835.19	16,222,023.97	850.3	10,228,991.33	3,033,594.88	187.0	29,751,774.10	8,026,969.22	557.8	5,206,220.59	1,668,106.77	124.6	2,388,086.16	Pennsylvania
Rhode Island	2,628,496.20	1,119,688.09	64.8	1,307,118.37	412,640.21	21.9	1,586,883.95	454,695.57	28.5	45,705.55	13,050.00	0.9	667,495.13	Rhode Island
South Carolina	11,163,347.84	5,121,267.54	1,235.9	1,345,670.54	504,380.82	71.3	7,472,425.62	3,330,372.90	377.4	435,848.55	177,606.98	33.2	667,895.76	South Carolina
South Dakota	12,091,434.67	5,989,879.00	1,447.9	4,468,181.03	2,162,037.09	596.0	3,521,652.42	1,680,790.49	554.8	89,856.30	44,928.15	18.0	1,289,155.27	South Dakota
Tennessee	13,789,140.98	6,732,079.77	497.9	6,210,452.31	2,761,448.99	211.9	7,868,816.05	3,661,034.15	280.4	1,246,282.97	594,167.54	38.0	1,531,860.55	Tennessee
Texas	54,120,970.83	21,057,940.12	3,907.1	12,224,766.29	5,120,935.54	736.6	19,738,316.97	8,825,157.77	1,114.9	3,067,800.41	1,379,032.93	121.6	4,223,364.64	Texas
Utah	6,259,159.41	3,818,836.91	423.1	1,034,736.98	636,446.44	52.1	2,630,523.20	1,896,195.08	225.2	32,351.34	23,939.99	0.1	1,443,360.58	Utah
Vermont	3,015,174.51	1,452,894.45	107.8	1,186,757.35	544,749.67	26.4	1,066,055.68	444,520.18	23.5	148,135.35	74,067.67	0.3	752,275.03	Vermont
Virginia	13,099,720.01	6,271,998.20	676.2	8,148,301.82	3,764,679.49	297.5	6,035,474.99	2,596,064.62	187.1	1,473,332.50	704,234.98	39.5	164,536.71	Virginia
Washington	13,352,504.18	6,117,211.87	526.7	3,591,201.52	1,607,897.59	137.2	2,525,208.65	1,232,400.00	27.6				1,188,266.54	Washington
West Virginia	7,343,200.86	3,230,293.33	326.7	1,553,622.90	632,072.94	54.5	5,669,701.47	2,261,429.37	143.9	370,896.16	142,956.79	14.0	1,085,758.57	West Virginia
Wisconsin	21,807,140.91	8,919,640.62	1,451.7	2,044,789.76	1,013,336.87	95.2	4,878,226.54	2,393,891.23	234.0	643,196.13	321,596.00	33.2	4,790,350.28	Wisconsin
Wyoming	8,809,819.33	4,739,096.67	982.0	1,845,944.24	1,126,568.38	145.7	2,632,505.19	1,658,909.53	166.0	91,395.64	58,675.00	7.3	983,024.42	Wyoming
Hawaii							1,050,897.93	312,635.18	15.9				787,517.82	Hawaii
TOTALS	740,140,790.82	325,654,346.00	41,898.3	168,826,367.15	75,992,051.60	8,099.5	342,999,099.22	146,435,969.24	14,015.5	47,146,908.88	18,019,076.54	1,990.9	105,273,566.62	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 86,434,928.71 Federal aid $ 36,787,228.02 Miles 3,325.3