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D. M. BEACH, *Editor*

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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SOIL DISPLACEMENT UNDER A LOADED CIRCULAR AREA[1]

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by L. A. PALMER, Associate Research Specialist, and E. S. BARBER, Junior Highway Engineer

THIS REPORT describes a procedure for evaluating the supporting characteristics of the subgrade under flexible types of pavement. The procedure uses laboratory-determined stress-deformation curves for the subgrade soil in conjunction with rational theoretical analyses. The method of approach does not include the making of penetration and loading tests directly on the subgrade, a procedure that has various disadvantages. A correlation between measured deflections of the subgrade under wheel loads and the deflections computed from laboratory test data and theory is an indicated experimental procedure that should extend knowledge in this field.

Various formulas for use in the design of flexible-type pavements have been advanced in the past, but the theories and assumptions upon which they have been based have not permitted of experimental verification. In a rational design, knowledge of the stress-deformation characteristics of the subgrade soil is essential. Such knowledge would make it possible to set up maximum allowable deformations of the base to serve as criteria for use in designing the surface.

This report presents a method of evaluating the supporting characteristics of the subgrade under flexible-type pavements, and, for the first time, suggests definite experimentation that can be made. It is felt that the suggested method has considerable promise and an extensive study is warranted to check the theory. Accordingly, a series of tests, including large-scale outdoor tests, is planned to substantiate the theory or to determine what modification may be necessary.

The system of stresses at any point within a semi-infinite, elastically isotropic body produced by a uniform load over a circular area at the surface has been determined by A. E. H. Love[2] and S. D. Carothers.[3] Formulas for the vertical displacement, V, at any point of the elastic body due to the surface load are given in various texts[4] dealing with the theory of elasticity. The use of such formulas involves knowledge concerning two elastic constants—Poisson's ratio, μ, and the modulus of elasticity, E. Test data as reported by Terzaghi[5] and others show that the deformations of soils under load are not characteristic of elastic materials. Hence, there is difficulty in applying the formulas based on the assumption of elastic properties.

The purpose of this report is to indicate a method of computing vertical displacement in soil due to a uniform load over a circular area by the use of triaxial compression test data. The vertical displacement or settlement so computed is that caused solely by lateral yield of the soil. It is assumed that this type of settlement, S_L, is completed prior to the occurrence of any settlement due to consolidation of the supporting soil. A modulus of deformation, C, is used in the formulas instead of the modulus of elasticity, E, and since it is assumed that the settlement, S_L, occurs at constant volume, μ is necessarily equal to $\frac{1}{2}$.

[1] Paper presented at the Twentieth Annual Meeting of the Highway Research Board, December 3, 1940.

[2] The Stress Produced in a Semi-Infinite Solid by Pressure on Part of the Boundary, by A. E. H. Love. Philosophical Transactions of the Royal Society, series A, vol. 228, 1929.

[3] Test Loads on Foundations as Affected by Scale of Tested Area, by S. D. Carothers. Proceedings, International Mathematical Congress, Toronto, pp. 527–549, 1924.

[4] See for example equations 203 and 204, page 335, of Theory of Elasticity, by S. Timoshenko. McGraw-Hill Book Company, first edition, 1934.

[5] Determination of Consistency of Soils by Means of Penetration Tests, by Charles Terzaghi. PUBLIC ROADS, vol. 7, No. 12, Feb. 1927.

The modulus of deformation, C, is herein defined as the ratio of stress to deformation without regard to the nature of the deformation whether it be elastic or plastic deformation or both.

The method of analysis used in this paper for the case of a uniform load over a circular area may be applied, with certain modifications, to the case of a parabolic or conical distribution of load over the circular area.

MODULUS C DETERMINABLE FROM STABILOMETER TEST DATA

In stabilometer tests, cylindrical soil samples encased in rubber sleeves are compressed to failure by applying a vertical load with or without lateral pressure. Lateral pressure is applied by air or fluid and is constant during an individual test. The decrease in length, Δh, of the sample with increasing vertical load may be measured by means of a micrometer dial attached to the moving plunger or by an automatic recording device. The initial length of the sample is designated as h.

A more complete description of plotting the stabilometer test data in connection with computing C has already been published.[6] In figure 1, the portions of the three curves for which the vertical pressure, v, is less than the lateral pressure, l, have been omitted and the coordinate axes have been moved to the right so that the point where $l=v$ falls on the $\frac{\Delta h}{h}=0$ axis.

In figure 1, the slope of any of the secant lines is taken as the modulus of deformation, C, within the range of loading indicated. The more nearly straight the line representing the v versus $\frac{\Delta h}{h}$ relationship, the more nearly the secant modulus becomes a tangent modulus. The greater the curvature of the line representing the v versus $\frac{\Delta h}{h}$ relationship, the greater is the divergence of the secant line from the curve. Usually this divergence increases with increasing load.

From the theory of elasticity, the vertical strain ϵ_z, at any point on the axis of loading in the stressed earth below the uniformly loaded circular area of radius, a, is

[6] The Settlement of Earth Embankments, by L. A. Palmer and E. S. Barber. PUBLIC ROADS, vol. 21, No. 9, November 1940.

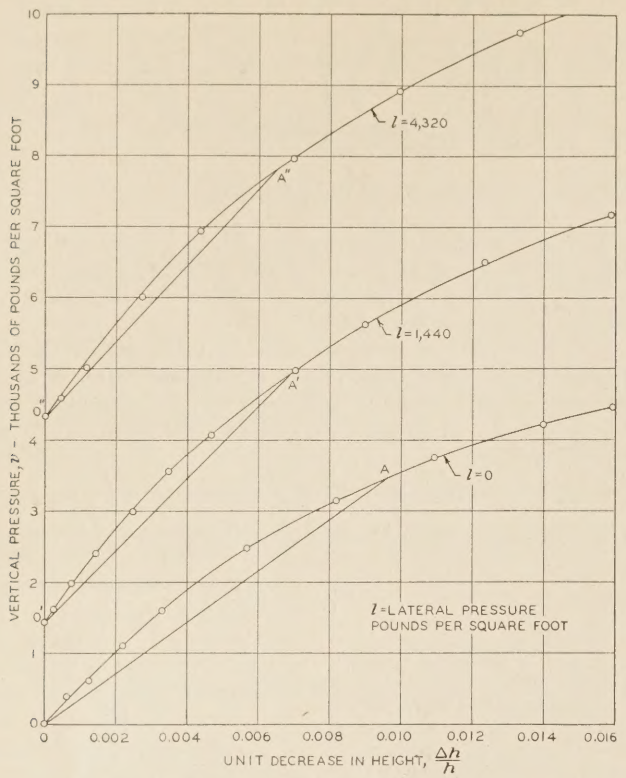

FIGURE 1.—LOAD-COMPRESSION TEST RESULTS.

$$\epsilon_z=\frac{\partial V}{\partial z}=\frac{1}{E}(p_z-2\mu p_r) \text{ --------------- } (1)$$

where z is the depth of the point; V is the vertical displacement at the point; and p_z and p_r are normal stresses in the vertical and radial directions, respectively, acting at the point.

If instead of E, the modulus of deformation, C, is used and taken as a constant, equation 1 becomes

$$\frac{\partial V}{\partial z}=\frac{1}{C}(p_z-2\mu p_r) \text{ --------------- } (2)$$

The expressions for the normal stresses, p_z and p_r, are

$$p_z=p\left[1-\frac{z^3}{(a^2+z^2)^{3/2}}\right] \text{ ---------- } (3)$$

and

$$p_r=\frac{p}{2}\left[1+2\mu-\frac{2(1+\mu)z}{(a^2+z^2)^{1/2}}+\frac{z^3}{(a^2+z^2)^{3/2}}\right] \text{ ------ } (4)$$

where p is the unit surface load. The origin of coordinates is taken at the center of the circular area of earth surface. By substituting equations 3 and 4 in equation 2 and integrating between the limits z and ∞, there results

$$\text{maximum } V=\frac{p}{C}\left[(2-2\mu^2)(a^2+z^2)^{1/2}-\frac{(1+\mu)z^2}{(a^2+z^2)^{1/2}}+(\mu+2\mu^2-1)z\right] \text{ -------------- } (5)$$

By taking $\mu=\frac{1}{2}$,

$$S_L=\frac{3pa^2}{2C(a^2+z^2)^{1/2}} \text{ -------------- } (6)$$

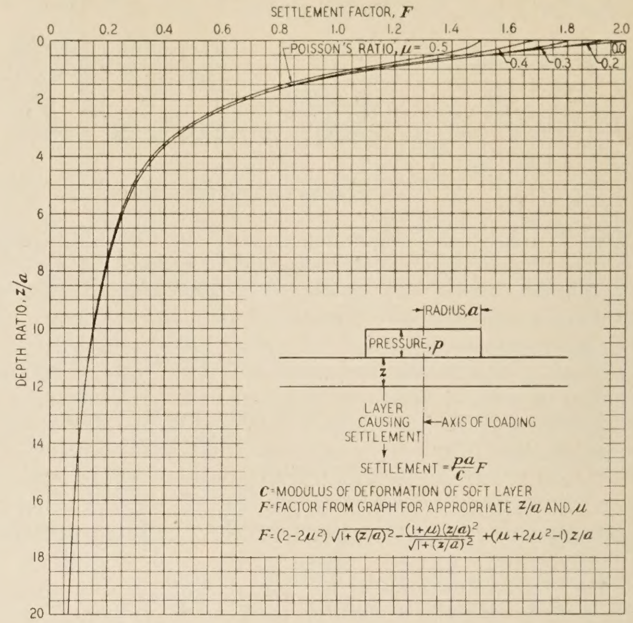

FIGURE 2.—SETTLEMENT UNDER CENTER OF UNIFORM CIRCULAR LOAD.

or in general,

$$\text{maximum } V=\frac{pa}{C}F \text{ ---------------- } (7)$$

where

$$F=(2-2\mu^2)\sqrt{1+\left(\frac{z}{a}\right)^2}-\frac{(1+\mu)\left(\frac{z}{a}\right)^2}{\sqrt{1+\left(\frac{z}{a}\right)^2}}+(\mu+2\mu^2-1)\frac{z}{a}$$

F may be called the "settlement factor." In figure 2, F is plotted against values of z/a for values of 0, 0.2, 0.3, 0.4, and 0.5 assigned to μ. It is observed that the value of μ has but little influence on the value of F if z/a exceeds unity.

SHAPE OF LOADED AREA HAS LITTLE EFFECT ON SETTLEMENT

In equations 5 and 6, z can have any value. For $\mu=\frac{1}{2}$ and $z=0$, equation 6 becomes

$$S_L=\frac{3pa}{2C} \text{ ------------------ } (8)$$

which is the total settlement due to lateral yield from the load down to infinite depth, and it also is the downward displacement of a soil particle at the center of the circular surface-contact area. Equation 8 gives S_L along the centerline. At points removed from the centerline, the settlement is less than this value. According to Timoshenko[4] the average settlement under the uniformly loaded circular area, which is the average deflection of all points over the circular contact area, is 85 percent of the maximum deflection at the center. Timoshenko[4] also shows that the average settlement under a uniformly loaded circular area is practically the same as that under a uniformly loaded square area. He shows further that the average settlement under a uniformly loaded rectangular area having sides of ratio 2:1 is about 4 percent less than that realized in the case of the circle or square. Thus the effect of shape of loaded area on settlement is not so important as might at first be thought.

[4] Theory of Elasticity, by S. Timoshenko. McGraw-Hill Book Co., first edition, 1934, pp. 338 and 339.

The depth, z, figure 2, may be considered as the thickness of an incompressible yet flexible layer, infinite in lateral extent, below which there is yielding soil of infinite depth. Such a condition may be realized for practical purposes if a bed of stabilized soil-aggregate is spread over a clay soil and a load is applied to the stabilized surface. The settlement in this case would be due entirely to yielding of the clay if the thickness, z, of the soil-aggregate bed is assumed to remain constant under load and to have no flexural strength. Then, as shown in figure 2, for given values of p and C, increasing the ratio, z/a, either by increasing z or decreasing a, reduces the settlement factor, F, and hence the value of S_L.

This reasoning is based on the simplifying assumption that the bed of material of thickness z and the more yielding material below it comprise a single homogeneous and isotropic mass of material insofar as the system of stresses is concerned. As pointed out previously,[7] a very similar assumption has been made and used to very good advantage in estimating the settlements of structures supported by one or more layers of compressible soil with one or more intervening layers of sand.

For a uniform load on a circular area at the surface, the maximum shearing stress at each point on the axis of loading is

$$s_{max}=\frac{1}{2}(p_z-p_r) \qquad (9)$$

where the difference, p_z-p_r, is the principal stress difference.

From equations 3 and 4,

$$p_z-p_r=p\left[\frac{1-2\mu}{2}+(1+\mu)\frac{z}{(a^2+z^2)^{1/2}}-\frac{3z^3}{2(a^2+z^2)^{3/2}}\right] \qquad (10)$$
$$=pf$$

where

$$f=\frac{1-2\mu}{2}+\frac{(1+\mu)\frac{z}{a}}{\sqrt{1+\left(\frac{z}{a}\right)^2}}-\frac{3}{2}\left(\frac{\frac{z}{a}}{\sqrt{1+\left(\frac{z}{a}\right)^2}}\right)^3$$

Values for the principal stress differences at different depths on the axis of loading and for different values of μ are shown in figure 3. It may be noted from this figure that for $\mu=\frac{1}{2}$, the maximum principal stress difference is equal to $0.58p$ and occurs at the depth, $z=0.71a$.

It is necessary to bear in mind that p_z-p_r is the difference between the vertical and lateral pressures on the axis of loading. Obviously there could be no lateral yielding of the supporting soil if p_z and p_r were equal in magnitude and of the same sign at all points.

In figure 1, the points A, A′, and A″ were selected so as to give $v-l$ the same value. In general, the modulus, C, obtained from the slopes of secant lines, such as OA, O′A′, etc., vary somewhat with the magnitude of the lateral pressure, l, maintained constant during a single test. Uusally the value of C is lowest for the curve, $l=0$. The procedure in this paper is to use an average value of C obtained from two or more curves of the type shown in figure 1 for a definite value

[7] Stresses Under Circular Loaded Areas, by L. A. Palmer. Proceedings of the Highway Research Board, vol. 19, 1939.

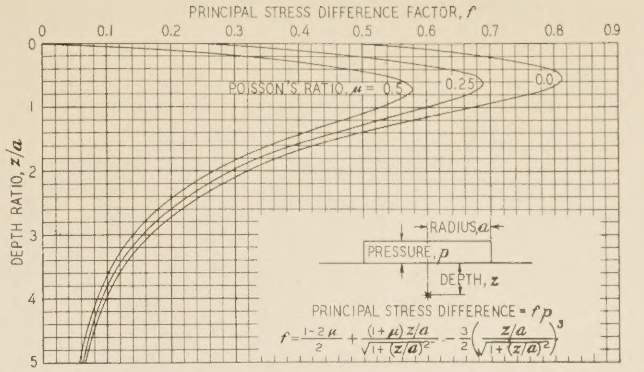

FIGURE 3.—PRINCIPAL STRESS DIFFERENCE UNDER CENTER OF UNIFORM CIRCULAR LOAD.

of $v-l$, applicable to the particular problem. The value of $v-l$ which determines the average C to be used is the maximum principal stress difference at any point within the yielding soil mass and on the axis of loading. Obviously this procedure is on the side of safety.

EXAMPLE ILLUSTRATES USE OF PRINCIPLES

The use of these principles may be illustrated by an example.

Assume the existence of a clay layer extending from the earth surface to an infinite depth. Assume there is a uniform load at the surface of 3 tons per square foot distributed over a circular area having a radius of 10 feet. Taking μ as $\frac{1}{2}$, the greatest principal stress difference on the axis of symmetry is at a depth of $0.71a=0.71\times10=7.1$ feet. Soil samples taken at this depth have the stress deformation characteristics shown in figure 1. The greatest principal stress difference$=0.58p=0.58\times6{,}000=3{,}480$ pounds per square foot.

On the curve, $l=0$, figure 1, for $v-l=3{,}480$, $v=3{,}480$ pounds per square foot. This is at point A on this curve. The secant line, OA is drawn.

The slope of OA is $\frac{3{,}480}{0.0096}$ or 362,000 pounds per square foot, the value of C from this curve. For the curve obtained with $l=1{,}440$ pounds per square foot in the triaxial test, the point A′ is determined by adding 1,440 to 3,480 to obtain a value, $v=4{,}920$ pounds per square foot. The corresponding percentage deformation is 0.68. Then C from this curve is $\frac{3{,}480}{0.0068}$ or 512,000 pounds per square foot. Similarly, from the secant line O″ A″ for the curve, $l=4{,}320$ pounds per square foot, C corresponding to $v=7{,}800$ pounds per square foot on the curve, is computed as $\frac{3{,}480}{0.0065}$ or 535,000 pounds per square foot. The average modulus is then 1/3 (362,000+512,000+535,000) or 470,000 pounds per square foot.

In this problem, the depth of a layer or zone wherein compression due to yielding does not occur is zero and hence z/a is zero. From figure 2, for $z/a=0$ and $\mu=1/2$, $F=1.5$. Then

$$S_L=\frac{pa}{C}F=\frac{6{,}000\times10}{470{,}000}\times1.5$$
$$=0.19 \text{ foot, or about 2.3 inches.}$$

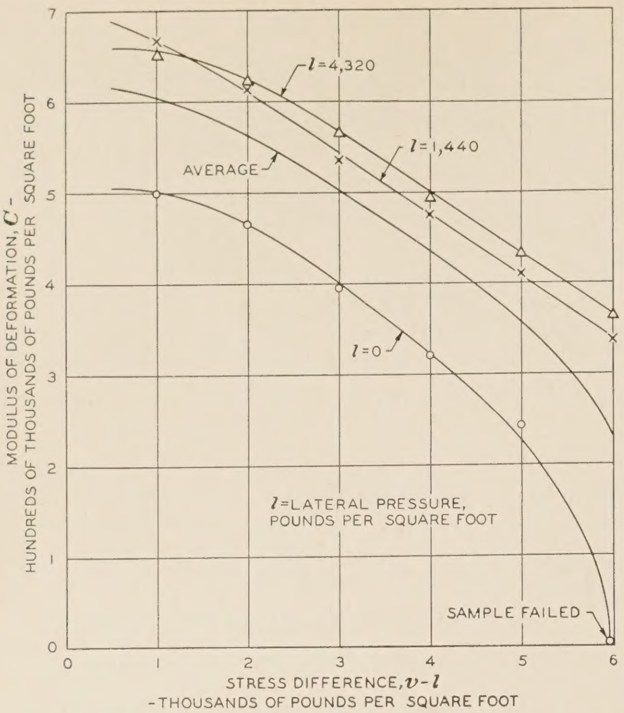

Figure 4.—Variation of Modulus of Deformation With Stress.

Suppose now that there is a bed of compact sand 5 feet thick at the surface and that the clay is below. Neglecting the lateral displacement or vertical compaction of the sand, with $z/a=\frac{5}{10}=0.5$, the displacement factor, figure 2, is seen to be reduced from 1.5 to 1.33 and the value of S_L is reduced to $2.3\times\frac{1.33}{1.5}=2.0$ inches. Similarly, if the bed of sand is 10 feet thick, z/a becomes 1 and F, figure 2, is 1.06.

With a bed of compact sand 10 feet thick, the greatest principal stress difference is in the sand and not in the clay. The greatest principal stress difference in the clay is then 10 feet down from the ground surface on the axis of symmetry and, from figure 3, its value at the point, $z/a=1$, is $0.53p$. For the soil data plotted in figure 1, the moduli of deformation corresponding to various $v-l$ values are shown in figure 4 for the three curves, $l=0$, $l=1{,}440$, and $l=4{,}320$ pounds per square foot. An average for these three curves is also shown. For $p=6{,}000$ pounds per square foot and a maximum principal stress difference of $0.53\,p=0.53\times6{,}000=3{,}180$ pounds per square foot, the corresponding C from the average curve, figure 4, is 490,000 pounds per square foot. Taking $F=1.06$ corresponding to $z/a=1$ and $\mu=\frac{1}{2}$,

$$S_L=\frac{6{,}000\times10}{490{,}000}\times1.06=0.13 \text{ foot or 1.6 inches.}$$

This is approximately 70 percent of the settlement without the sand, assuming no lateral displacement or compaction of the sand.

Actually, the sand would undergo some lateral displacement or compaction or both but to a considerably lesser extent than would a soft clay. Greatest settlements due to lateral yield are to be expected in plastic soils in which pore pressures of considerable magnitude are developed by loading.

With reference again to figure 4, consider a wheel load producing 10,000 pounds per square foot pressure on the surface of the clay, assuming a balloon tire and an equivalent radius of 6.5 inches for the contact area. Here the greatest principal stress difference on the axis of loading is $0.58\times10{,}000$ or 5,800 pounds per square foot, corresponding to an average C value, figure 4, of 270,000 pounds per square foot. For $z/a=0$, $F=1.5$ and

$$S_L=\frac{10{,}000\times6.5}{270{,}000}\times1.5=0.36 \text{ inch.}$$

Suppose now that the wheel rests on a flexible-type pavement of thickness 6.5 inches and that the lateral movement and compaction of pavement material under the wheel load is negligible. Under the pavement is the same clay, extending to an infinite depth. The maximum principal stress difference in the clay is $0.53p$ for $z/a=1$ and $\mu=\frac{1}{2}$, and is $0.53\times10{,}000=5{,}300$ pounds per square foot. The corresponding C value, average curve, figure 4, is 330,000 pounds per square foot and $F=1.06$, figure 2. Then

$$S_L=\frac{10{,}000\times6.5}{330{,}000}\times1.06=0.21 \text{ inch.}$$

By making similar computations for the same soil and for various values of z, the settlements shown in table 1 are obtained. It is observed in table 1 that when the pavement thickness is of magnitude such that z/a is equal to or greater than 2, any further increase in pavement thickness effects a relatively small decrease in S_L. It is also noted that the modulus, C, increases rapidly as the principal stress difference decreases, a fact that is indicative of the curvilinear relationship between v and $\frac{\Delta h}{h}$.

Table 1.—*The effect of pavement thickness on the estimated settlement, S_L, due to yielding of the subgrade. Unit load=10,000 pounds per square foot, equivalent radius of wheel load=6.5 inches and $\mu=\frac{1}{2}$*

Thickness of pavement, z	$\frac{z}{a}$	Maximum principal stress difference in subgrade, $v-l$	Modulus of deformation, C	Deflection factor, F	Settlement, S_L
Inches		*Pounds per square foot*	*Pounds per square foot*		*Inches*
0	0	5,800	270,000	1.50	0.36
3.25	0.50	5,800	270,000	1.33	.32
6.50	1.00	5,300	330,000	1.06	.21
8.00	1.23	4,600	390,000	.93	.16
10.00	1.54	3,750	455,000	.80	.11
12.00	1.85	2,950	505,000	.71	.09
15.00	2.31	2,200	550,000	.59	.07
18.00	2.77	1,600	580,000	.51	.06

The effect of moisture content and compaction on the modulus, C, of a typical clay soil is shown in table 2. The settlement, S_L, caused by lateral displacement of this clay when subjected to a uniform load over a circular area is also shown. For the computations shown in table 2, μ is taken as $\frac{1}{2}$ and the surface load is assumed to be applied directly to the clay; that is, $z/a=0$.

The effect of moisture content and density is strikingly illustrated by the computed values in table 2.

(Continued on page 198)

EFFECTS OF HIGHWAY LIGHTING ON DRIVER BEHAVIOR

BY THE DIVISION OF HIGHWAY TRANSPORT, PUBLIC ROADS ADMINISTRATION

Reported by W. P. WALKER, Assistant Highway Engineer-Economist

Limited data indicate that the behavior of drivers operating under artificial light conforms very nearly to their behavior in the daytime, but that the behavior of drivers at night without overhead lights differs measurably from their behavior in the daytime. Differences between the behavior of drivers during daylight and darkness are most apparent in the frequency of passing and in the transverse positions of vehicles on the pavement. There is inconclusive evidence that speed may also be affected.

During daytime the drivers utilized 57.7 percent of the available opportunities for passing as compared to 55.6 percent during nighttime with the highway lighted. At night with the highway unlighted the drivers utilized only 38.5 percent of the available opportunities for passing.

The frequency distributions of transverse positions are almost identical for conditions of daytime and nighttime with the highway lighted, but there is a marked difference in these distributions for conditions of daytime and nighttime with the highway unlighted. The average position of the right wheel of passsenger cars moving freely was 3.3 feet from the edge of a 20-foot pavement during both daytime and nighttime with the highway lighted. With the highway unlighted, this average position was ½ foot nearer the center of the road.

ACCIDENT RECORDS have consistently shown that nighttime driving is more hazardous than daytime driving. During recent years illumination engineers have performed considerable research on highway lighting with the object of reducing the ratio of nighttime to daytime accidents per vehicle-mile of travel. Several hundred miles of rural highway are now lighted, many of these being temporary installations for purposes of demonstration and experimentation. One such installation is a 1-mile section on U. S. Route 422 near Chagrin Falls, Ohio. Excessive grade and curvature at this location result in its being considered highly hazardous, and for this reason it was selected by the Nela Park Engineering Department of the General Electric Company for study of methods of illumination and measurement of factors affecting visibility on lighted roads.

In the fall of 1939, the Public Roads Administration and the Ohio Department of Highways conducted studies at this location in an effort to determine the effect of lighting on driver behavior. These studies were made over a 5-day period and three types of equipment were employed, each designed to obtain different information regarding driver behavior. With this equipment, comprehensive data were collected on passing practices, transverse positions of vehicles, and vehicle speeds and spacings. The primary objective was to determine to what extent driver behavior varied in daytime, in nighttime with the road lighted, and in nighttime with the road unlighted. Inclement weather during the observations increased the number of variables to include conditions of wet and dry pavement.

The highway approaches the section on a tangent with a slightly undulating grade. About ¼ mile from the first light and within the lighted section, the road rises slightly, then drops sharply on a grade of about 10 percent for a distance of approximately ½ mile. There are two horizontal curves on this grade, one of them being very sharp. The lower ¼ mile of the section is approximately level tangent. Figure 1 is a sketch of the plan and profile showing the operating positions of the study equipment.

The surface is portland-cement concrete pavement in fairly good condition, having a width of 20 feet except on and below the hill where it is 27 feet wide. The shoulders on the 20-foot section are 10 feet wide and consist of 2 feet of clay-gravel and 8 feet covered with grass and in good condition. The section 27 feet wide has a 6-inch curb on each side. Lighting is by means of incandescent lamps in specially designed reflectors mounted 25 feet high, 125 feet apart, and extending 5 feet out over the pavement. The lights are along only one side of the road. Figure 2 shows a portion of the section in the daytime and when lighted at night.

In evaluating the effects of highway lighting from a safety standpoint the accident record itself would be the most desirable index, but, since the number of accidents per mile of highway is relatively small, to obtain reliable data for a 1-mile section of highway would require years. Moreover, no accident records were available for this particular section of road. As a substitute for accident records it is possible, by a critical examination of driver behavior under the various conditions, to judge the probable effects of these conditions. From the results of driver-behavior studies, it is possible to find instances where a driver on an unlighted highway was unquestionably driving too fast or in an otherwise reckless manner, but there is no way of proving that the particular driver might not drive just as recklessly during daylight or on a lighted highway. However, the chances are that there will be just as many reckless drivers using the highway at night when it is not lighted as when it is lighted.

A differentiation between safe and unsafe driving practices under any set of driving conditions is difficult to make. Undoubtedly, the safest driving conditions exist during hours of daylight with a dry pavement,

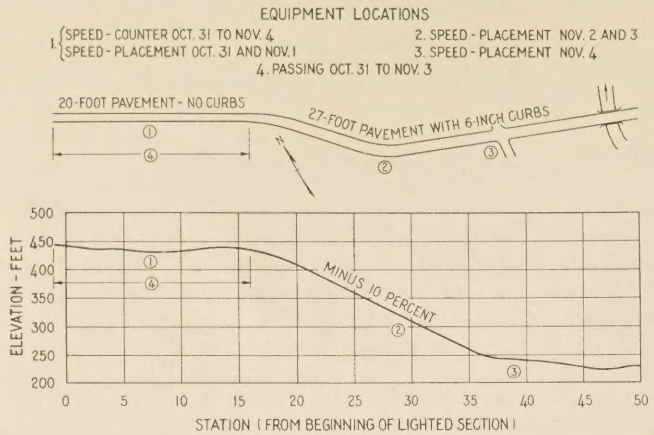

Figure 1.—Plan and Profile of One-Mile Section of Lighted Highway on U. S. Route 422 Near Chagrin Falls, Ohio.

and the average driver under these conditions might be expected to perform in a somewhat different manner with respect to speed, distance from car ahead, and transverse position on the pavement than he would on the same highway after dark. If the drivers perform on a lighted highway at night in the same manner they perform on the highway in daylight, it is safe to assume that the vehicles are moving with greater safety and facility than they would on an unlighted highway. The degree to which driver performance on the lighted highway at night approaches that for daylight hours should be a measure of the effectiveness of the lighting in bringing about safer driving conditions.

PASSING PRACTICES STUDIED ON LIGHTED SECTION

Equipment for determining the passing practices of motor-vehicle drivers has been developed and its use described.[1 2] The equipment permits determination of the speed and time spacing of each vehicle at any point within a half-mile section, and shows whether the vehicle was in its own or the opposing lane of traffic or was straddling the centerline of the road. It does not permit a determination of the pavement edge clearances of vehicles.

Because of topographic conditions on the section of road studied, it was practical to install only two-thirds of the detector tubes, the location selected being on the tangent at the top of the hill. On a portion of this study section passing was restricted by inadequate sight distance. Because of this and the low traffic volumes prevailing, the number of passing maneuvers recorded was not great. The equipment was operated during afternoons and evenings until about 10 p. m. for 4 days. The lights were off on alternate evenings.

Table 1 shows the actual operating time of the recorders, the number of vehicles, and the number of passing maneuvers recorded under the various conditions studied. Of the 107 passing maneuvers recorded, a complete record was obtained of only 53, the other 54 having been started before entering or completed after leaving the study section. In all 107 cases, however, the passed and passing vehicles were recorded while abreast of each other so that data were obtained on at least half of each maneuver.

[1] Procedure Employed in Analyzing Passing Practices of Motor Vehicles, by E. H. Holmes, PUBLIC ROADS, vol. 19, No. 11, January 1939.
[2] Progress in Study of Motor-Vehicle Passing Practices, by O. K. Normann, PUBLIC ROADS, vol. 20, No. 12, February 1940.

Table 1.—*Results of studies using passing equipment*

Condition	Net hours studied	Vehicles recorded	Passings recorded
Daylight	1.85	448	41
Night—lights on	3.77	496	20
Night—lights off	4.48	616	20
Twilight	1.50	462	26
Total	11.60	2,022	107

Conclusive results obviously cannot be drawn from such a small and varied sample. It is of interest, however, to examine a few of the passing maneuvers that were made under what might be considered hazardous conditions. Since no passing maneuver was made when an oncoming vehicle was so near as to constitute a hazard, this sample includes only passing maneuvers that were made where the driver could see less than 400 feet of road surface ahead of him. This figure is used because it represents the sight distance at the point of beginning of a double white line center marking. The remainder of the passing study section had no centerline marking except the black center joint.

Eighteen vehicles started to pass where the sight distance was 400 feet or less and these maneuvers are shown graphically in figure 3. These data are presented merely as a matter of general interest, since any comparison in numbers would be inconclusive and the similarity of the passing maneuvers permits of little differentiation for various conditions. It will be noted that, under all four conditions of light, there are cases where the passed and the passing vehicles were abreast of one another at points where the sight distance was only 200 feet. The speeds of these passing vehicles varied between 25 and 50 miles per hour. Had an oncoming vehicle made its appearance during one of these maneuvers, either the passing or oncoming vehicle would almost certainly have been required to take refuge on the shoulder to avoid a collision. In these 18 passing maneuvers, 11 of the passed vehicles were trucks, busses, or tractor-semitrailer combinations moving relatively slowly.

Several of the passing maneuvers of westbound vehicles (fig. 3) were accomplished without creating any traffic hazard, the reason being that the sight distance increased to about 2,000 feet before the passing vehicle was completely in the left lane. These were violations of the center striping that would not be so classified had the marking been of the directional type which permits passing in one direction while prohibiting it in the opposite direction. The number of vehicles that could have passed but were discouraged from doing so by reason of the center striping cannot be determined.

Table 2 shows the relationship between the actual number of passing maneuvers recorded and the number that could have been accomplished under favorable conditions. Classified as "potential" passings are those cases where a vehicle was following another vehicle at a spacing of 1½ seconds or less at a point where no restriction was offered to passing by an oncoming vehicle within 1,500 feet or by a sight distance less than 1,200 feet. The percentage that these potential passings are of the total is shown in the last column. These figures show that 42.3 percent of these drivers were reluctant to pass during daylight as compared to 61.5 percent at night when the highway was unlighted. When the highway was lighted, however, only 44.4 percent of the drivers preferred to follow the vehicle ahead rather than

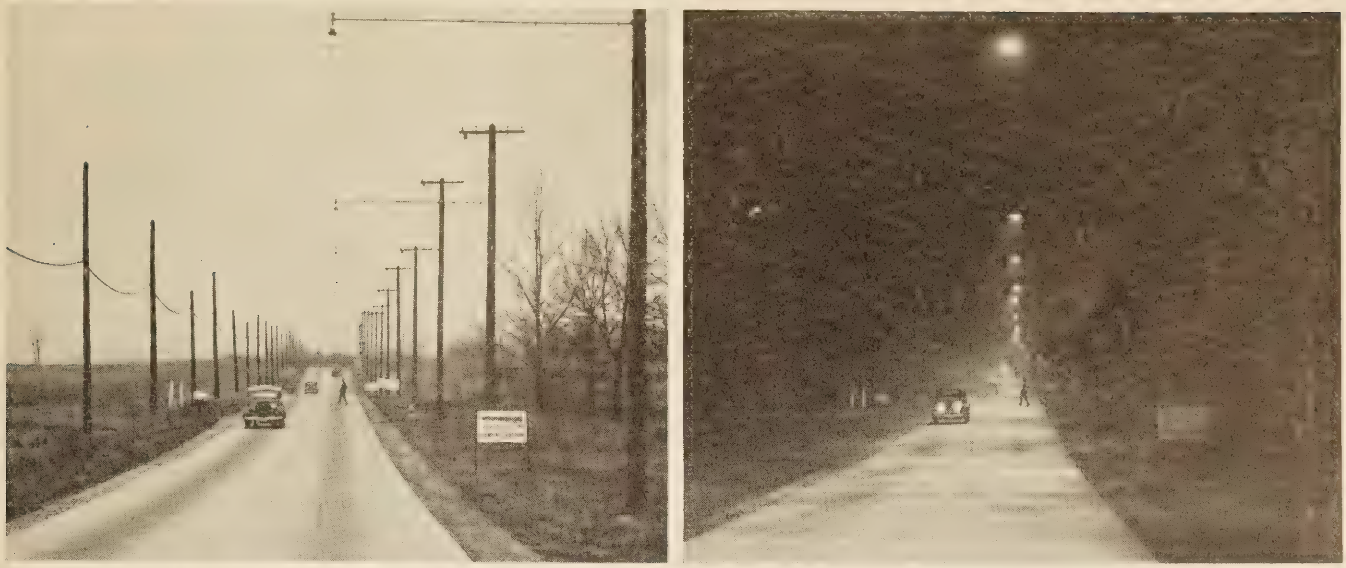

FIGURE 2.—APPEARANCE OF A PORTION OF THE SECTION IN DAYLIGHT AND WHEN LIGHTED AT NIGHT.

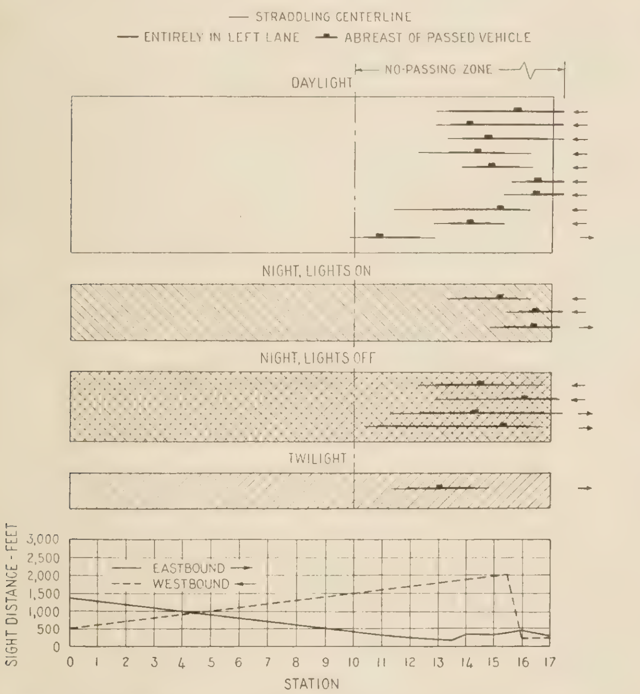

FIGURE 3.—PASSING MANEUVERS MADE IN VIOLATION OF CENTER-STRIPED NO-PASSING ZONE.

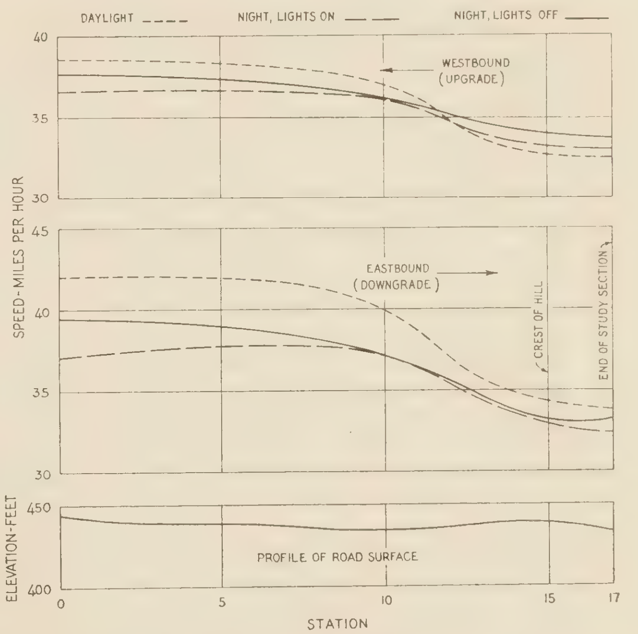

FIGURE 4.—AVERAGE SPEEDS OF ALL VEHICLES AS RECORDED BY PASSING EQUIPMENT.

pass it. This compares favorably with the data for daylight conditions. It might be concluded from this that the drivers using the highway while it was unlighted were more cautious than those using the highway while it was lighted, but the important fact shown here is that the driving practices observed while the lights were on conformed much more nearly to those for daylight conditions than did the driving practices on the unlighted highway.

SOME HAZARDOUS DRIVING FOUND UNDER ALL CONDITIONS OF LIGHTING

The passing study equipment is well adapted to studying the variations in speed of vehicles over a length of highway. Such an investigation is of interest here to determine what effect the combination of a large diamond-shaped "Hill" sign and a flashing danger signal, both located near the crest of the rise just preceding the steep descent, had upon the speeds of vehicles under various conditions. In figure 4, the average speeds of vehicles as maintained throughout the section are plotted. The warning signs were located opposite station 14, facing eastbound traffic.

TABLE 2.—*Relations between the actual and potential number of passings under various conditions*

Condition	Number of passings			Percentage that potential is of total
	Actual	Potential	Total	
Daylight	41	30	71	42.3
Night—lights on	20	16	36	44.4
Night—lights off	20	32	52	61.5
Twilight	26	25	51	49.0

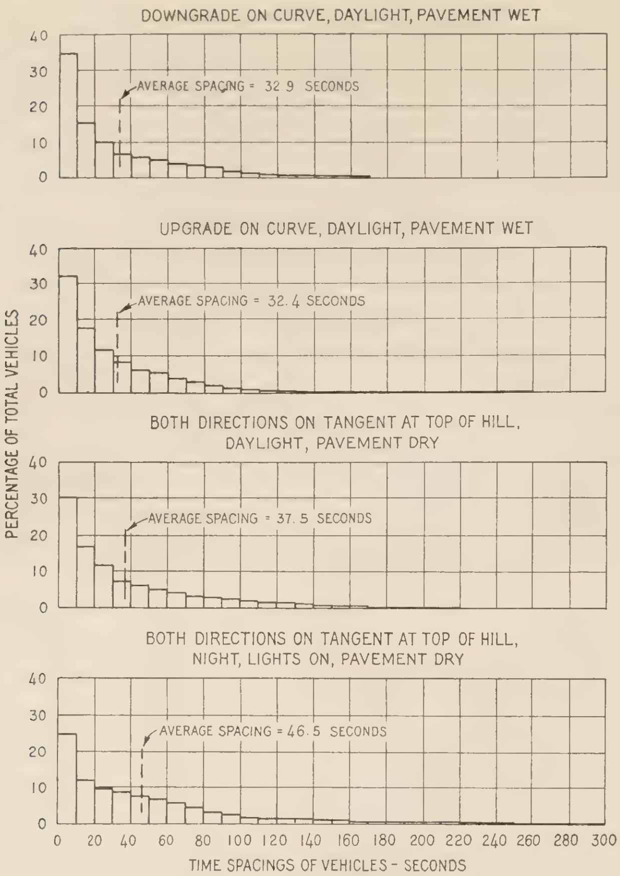

FIGURE 5.—FREQUENCY DISTRIBUTIONS OF TIME SPACINGS.

The speed curves for eastbound vehicles vary considerably for the three conditions shown. The average speed of the vehicles on the lighted highway showed a slight increase for the first 850 feet, whereas with the lights off, the average speed began to decrease almost immediately after entering the section. A number of explanations of this difference in behavior suggest themselves but none has any plausible basis. For each condition of lighting, the average speed showed a noticeable decrease upon approaching the warning signs, the amount of decrease varying from 5 miles per hour when the highway was lighted to 8 miles per hour during daylight. Without these warning signs, drivers unfamiliar with the road would be unaware of approaching any danger since the terrain visible from this point did not reveal the hill.

The ordinate of figure 4 for westbound vehicles represents their average speeds as they ascended the grade. The normal speed of these vehicles was somewhat lower than for the eastbound traffic.

The results of the passing study show that there are certain drivers whose hazardous driving habits cannot be corrected by means of artificial lighting, since such drivers are present under all conditions of light and darkness.

The results further show that there is a marked difference between the normal behavior of drivers during daylight and darkness, and that the behavior of drivers under artificial light conforms more nearly to their behavior during daylight than it does to their behavior during darkness.

The effect of weather conditions on driver behavior was more noticeable in the results of the speed-placement study than in the passing study. For this reason the results for various lighting conditions have been further classified to show variations caused by wet pavement.

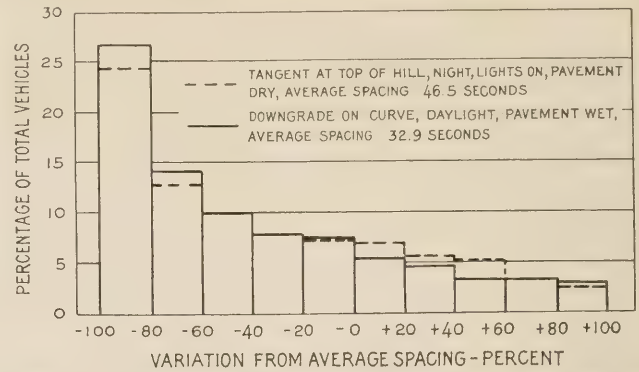

FIGURE 6.—FREQUENCY DISTRIBUTION OF VARIATIONS IN TIME SPACINGS FROM THE AVERAGE SPACING.

The passing equipment and placement equipment are two distinct sets of apparatus and are operated independently of each other. Speed and time-spacing data for all vehicles are an incidental part of the passing study records, and are available for any point within the study section. These data are also obtained by the speed-placement recorders but at only one point. In addition, the latter equipment records the positions of vehicle wheels with respect to the edge of the pavement.

Table 3 shows how the 32.49-hour net operating time of the speed-placement equipment was distributed with respect to weather and lighting conditions. This was the total time of study at three locations: One on the level tangent at the top of the hill, one on the sharp curve about midway of the steep grade, and one on the level tangent at the foot of the hill.

TABLE 3.—*Net hours of operation and vehicles recorded in speed-placement study*

Condition	Hours studied	Vehicles recorded
Daylight:		
Wet pavement	2.00	441
Dry pavement	10.13	2,388
Night—Lights on:		
Wet pavement	3.72	547
Dry pavement	5.92	1,131
Night—Lights off:		
Wet pavement		
Dry pavement	6.92	1,027
Twilight	3.80	1,225
Total	32.49	6,759

The frequency distribution of time spacings was investigated as a possible index of driving habits under various conditions of lighting and alinement. It is of interest that the time-spacing patterns varied only slightly from patterns found in previous studies, confirming the results of nearly all earlier studies. Under all conditions the percentage of vehicles traveling at or below the average time spacing was between 63 and 67. Earlier studies had showed invariably that approximately two-thirds of the vehicles traveled at or less than the average time spacing.

In figure 5, the frequency distributions of time spacings are shown for four conditions. Distributions for other conditions could also be shown but the similarity is so pronounced that further illustration is unnecessary.

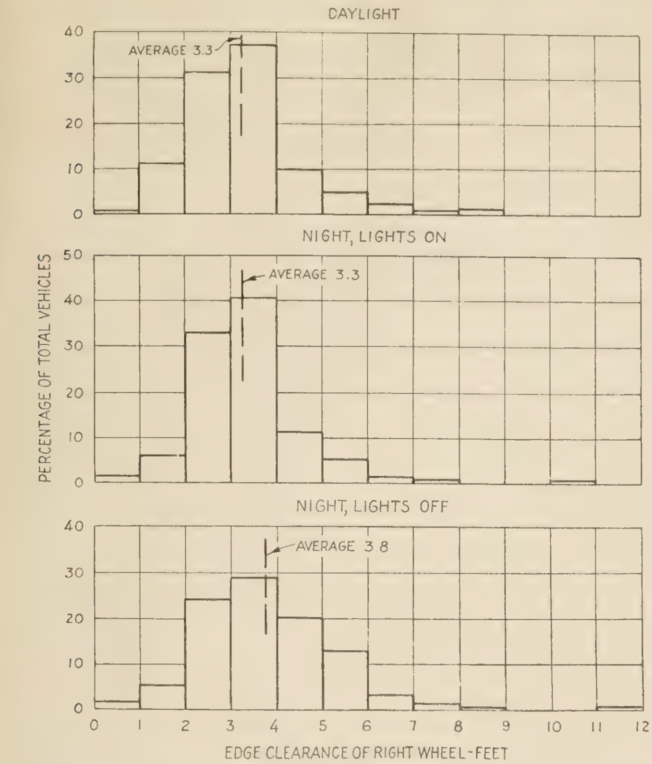

FIGURE 7.—EDGE CLEARANCES OF PASSENGER CARS MOVING FREELY ON TANGENT AT TOP OF HILL, PAVEMENT DRY.

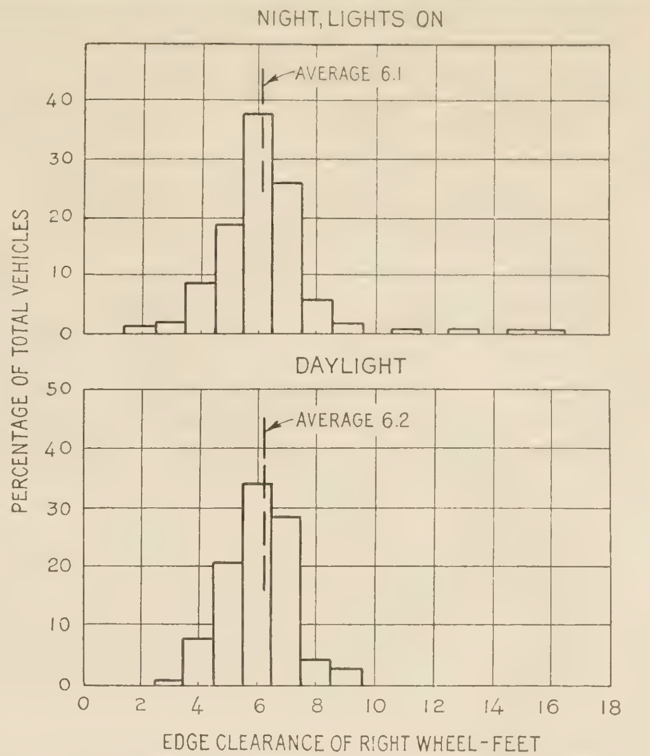

FIGURE 8.—EDGE CLEARANCES OF PASSENGER CARS MOVING FREELY ON TANGENT AT FOOT OF HILL, PAVEMENT DRY.

The fact that the frequency distribution of time spacings is a definite function of the average spacing and hence of the traffic volume, is more apparent in figure 6, where the distribution is based on the percentage of the average spacing. Here the distribution of spacings for one condition is superimposed on that for another up to twice the average spacing. These two conditions represent the extremes in traffic volumes studied. These results show that for the traffic volumes studied the time spacing of vehicles is independent of alinement, weather, and light conditions.

PLACEMENT OF VEHICLES UNDER VARIOUS CONDITIONS COMPARED

Data on the average placement of all vehicles with respect to the edge of the pavement are useful in comparing driver behavior under various conditions. In order to eliminate insofar as possible all extraneous factors, however, the most significant placement data are those obtained while drivers were uninfluenced by the presence of a preceding or an opposing vehicle. The edge clearances of such "freely" moving passenger cars on the tangent at the top of the hill, figure 7, show frequency distributions for conditions of daylight that are very similar to those at night with the highway lighted. In both cases the average edge clearance was 3.3 feet.

The distribution of edge clearances at night with the highway lights off, however, follows a noticeably different pattern and the average placement is ½ foot nearer the center of the road. With the highway lights on, 75 percent of the drivers followed a path not more than 2 feet wider than the car, the right wheel always being between 2 and 4 feet from the pavement edge. When the highway lights were off, the same percentage of drivers had a 3-foot variance in their path, the position of the right wheel being between 2 and 5 feet from the pavement edge. On the tangent at the foot of the grade the similarity between the placements during daylight and at night with the highway lights on is almost as striking (fig. 8). The difference in the average placement for these two conditions is only 0.1 foot. No record was obtained at this location at night with the highway unlighted.

Because of differences in weather conditions no direct comparison can be made of placement data recorded on the curve. The pavement was wet when studied with the highway lights on, and dry when studied with the lights off. Furthermore, the paths of vehicles traveling upgrade were restricted by the natural tendency of of drivers to hug the inside of the curve regardless of weather or lighting conditions. For the drivers traveling downgrade there is greater freedom in selecting the path which the driver feels is consistent with safety and comfort. Figure 9 shows that, for vehicles traveling downgrade, there was a slight difference between the placement distributions at night with the highway lights on and in the daytime under similar weather conditions. However, when the highway lights were off there was a marked difference in the distribution when compared to that for daylight with dry pavement.

As mentioned previously, the speeds of all vehicles were obtained as an incidental feature of the passing study at the top of the hill, and at three points with the speedmeter: The top of the hill, below the hill, and on the grade at the curve.

From the results of the passing study, figure 4, it appears that the speeds of vehicles, particularly of those just entering the lighted section, may not be representative of normal driving practice on lighted highways. This may be caused by the fact that the passing-study section was located at one end of the lighted portion of the highway, and drivers entering the section had had

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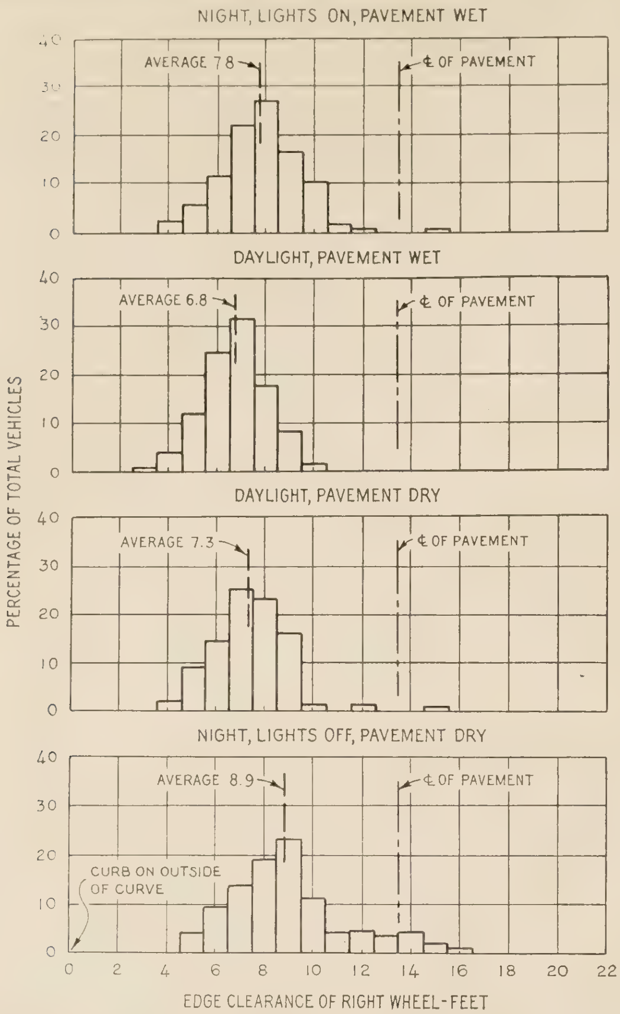

Figure 9.—Edge Clearances of Passenger Cars Moving Freely Downgrade on Curve.

no opportunity to adjust their driving to the changed condition. This assumption appears reasonable since the speeds of westbound vehicles were fairly uniform, as shown in figure 4.

At the first location of the speedmeter, 700 feet from the end of the lighted section, this same effect could be expected to influence the speed distribution. In addition, during study at this station the sample obtained under each condition was too small to indicate reliably the effect of illumination on speed distribution. The results obtained at the other two stations, located nearer the center of the lighted section, are not subject to these limitations.

Figure 10 shows that the distribution of speeds on the tangent below the hill was about the same under illumination as during daylight. The average speeds were 43 and 44 miles per hour, respectively. The posted speed limit on this road was 35 miles per hour, but these speed distributions show that 82 percent of the vehicles traveled in excess of this speed, both during daylight and at night with the highway lighted. Under both conditions 20 percent of the vehicles traveled in excess of 50 miles per hour.

On the curve, speed distributions were more varied in character, as shown in figure 11. For vehicles going downgrade the greatest similarity in speeds seems to exist between daylight with wet pavement and night without lights but with dry pavement. The speed dis-

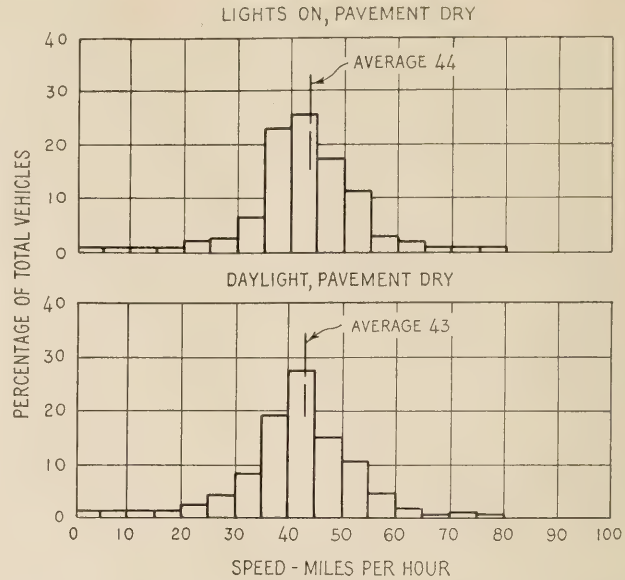

Figure 10.—Frequency Distributions of Speeds on Tangent at Foot of Hill.

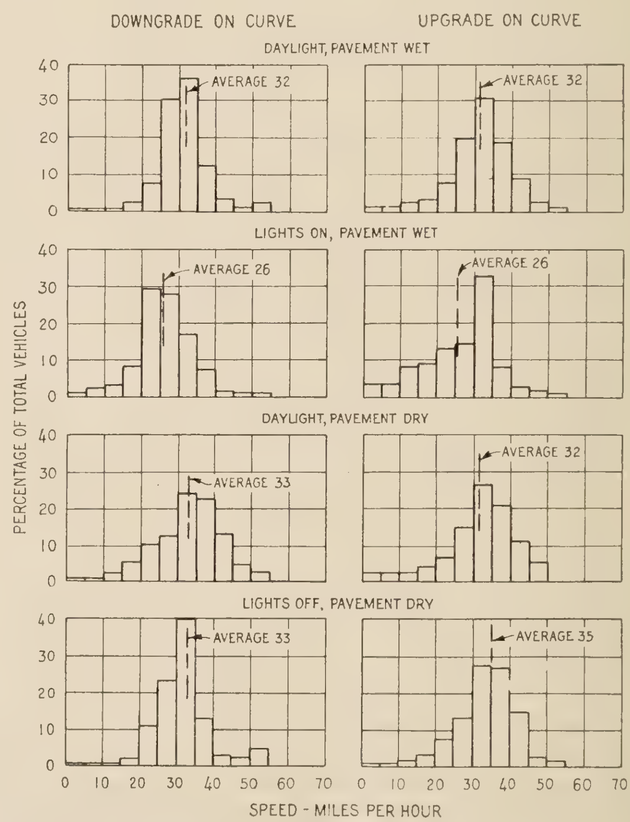

Figure 11.—Frequency Distributions of Speeds of All Vehicles.

tribution for vehicles on lighted, wet pavement seems to be in a class by itself, the average speed of 26 miles per hour being 6 miles per hour less than for daylight under similar weather conditions. For vehicles going upgrade, there are no marked differences in the patterns of speed distribution for the various conditions, but it is of interest that the average speeds of vehicles going upgrade under the various conditions are almost identical with the speeds of vehicles going downgrade under those same conditions.

(Continued on page 199)

EFFECT OF THE CHEMICAL PROPERTIES OF SOIL FINES ON THE PERFORMANCE OF SOIL-AGGREGATE MIXTURES

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by PAUL RAPP, Associate Chemist, and JACOB MIZROCH, Junior Chemical Engineer

AN ACCUMULATION of data from laboratory tests and field observations during the past several years has shown a general relation between the physical properties of soils and their performance in base and surface courses. The sampling and laboratory test procedures used in determining the physical properties of soils are presented in detail in the Standard Specifications for Highway Materials and Methods of Sampling and Testing of the American Association of State Highway Officials. The specifications [1] for base- and surface-course materials place special emphasis on the liquid limit, plasticity index, and mechanical analyses of the soil mortar or soil fines.

In general, these physical tests have been a satisfactory means of determining the suitability of soil-aggregate mixtures for road-building purposes, but in certain instances they have not been adequate for purposes of differentiation between satisfactory and unsatisfactory materials. The purpose of this paper is to indicate the additional information that may be derived from a study of the chemical properties of the soil fines which, together with the knowledge of physical properties, provides a more adequate basis for determining suitability than do physical properties alone.

CHEMICAL PROPERTIES OF SOILS DEPENDENT ON CLAY MINERALS PRESENT

The chemical properties of soils are dependent directly on the kind and amount of clay minerals present. Hence a brief resumé of the chemistry of clay minerals is given.

Hendricks and Alexander [2] have reported that clay minerals may be included in three general classes or groups as follows:

1. The kaolin group:
   - a. Kaolinite
   - b. Nacrite
   - c. Dickite
   - d. Halloysite

   composition $Al_2O_3.2\ SiO_2.2\ H_2O$.
2. The montmorillonite group:
   - a. Montmorillonite
   - b. Beidellite
   - c. Nontronite
   - d. Magnesium bentonite

   composition $Al_2O_3.4\ SiO_2.XH_2O$.
3. The hydrous micas. The general formula is

$$2K_2O.3R'O.8R''_2O_3.24Si_2O.12H_2O$$

where R′ denotes a divalent metal, magnesium for example, and R″ denotes a trivalent metal such as aluminum.

X-ray studies of minerals of the groups listed have shown that their constituent molecules, silica and alumina, are arranged in layers. These layers constitute the crystal lattice of a mineral.

In the kaolin group, the layers of silica and alumina alternate, forming a lattice with a molecular ratio of 2 silica to 1 alumina. Owing to replacement of silicon by aluminum within the lattice, ratios of less than 2 to 1 have been observed. Iron may replace aluminum in the lattice without affecting the molecular ratio.

The montmorillonite group is characterized by a lattice consisting of a layer of alumina between two layers of silica. Here the molecular ratio is 4 silica to 1 alumina. Replacement of silicon by aluminum may reduce the ratio to 3 to 1 in certain members of the group, without altering the lattice structure. Iron and magnesium may be present, substituted for the aluminum.

The term "hydrous micas" is a tentative one which serves to designate a group of related minerals having a theoretical formula similar to that of mica. However, these minerals contain less potassium and more water than true micas.

In addition to the clay minerals, certain accessory minerals are to be found in soils. Iron is present in the form of hematite, Fe_2O_3, and goethite, $FeO(OH.)$ Titanium has been found to be present in the form of the mineral, leucoxene, and as the oxide, rutile, TiO_2. Aluminum may be present in the form of diaspore or bohmite, $AlO(OH)$, and gibbsite, $Al(OH)_3$. Silica, SiO_2, may be present in the form of quartz or it may be present in an amorphous state.

Silica-sesquioxide ratio is defined as the molecular ratio of silica (SiO_2) to the combined oxides of iron and aluminum ($Fe_2O_3+Al_2O_3$). Silica-alumina ratio is the molecular ratio of silica to alumina (Al_2O_3). In this study both ratios were found to have a similar relation to physical properties. Therefore, the silica-alumina ratio is not discussed but is included in the paper as a matter of record. Silica-sesquioxide and silica-alumina ratios were determined in the colloidal clay fraction of the soil. This portion of the soil is considered the most active chemically. Usually it is highly weathered and consists mainly of one or more clay minerals of definite chemical composition. Silica-sesquioxide and silica-alumina ratios in the colloidal clay fraction of soils depend largely on the kinds and proportions of clay minerals present.

The colloids are extracted from the soils by means of a supercentrifuge. One hundred grams of soil passing the No. 10 sieve (or 50 grams of clay) are stirred to a smooth slurry with distilled water. The slurry is placed in a disperser and 1,000 milliliters of distilled water containing 1 milliliter of 5N ammonia are added. After dispersing for 2 minutes, the mixture is poured on a No. 270 sieve. The material retained on the sieve is dispersed in the same manner as before and the process continued until the sand remaining on the sieve is

[1] Standard Specifications for Materials for Stabilized Base Course, M 56-38 and Standard Specifications for Stabilized Surface Course, M 61-38, American Association of State Highway Officials.

[2] Minerals Present in Soil Colloids, by S. B. Hendricks and L. T. Alexander. I. Descriptions and Methods of Identification. Soil Science, vol. 48, No. 3, 1939, p. 258.

washed free of all colloid coating. The combined suspensions containing the soil fraction passing the No. 270 sieve are passed through a supercentrifuge. The centrifuge speed and rate of feed are so adjusted [3] that all particles coarser than 0.001 millimeter are thrown out of suspension inside the centrifuge bowl. The suspension passing through the centrifuge is concentrated with fine-grade Pasteur-Chamberland filter candles using suction. The material thrown out of suspension is removed from the bowl and redispersed in the filtrate from the candles. The resulting suspension is again centrifuged. The process is repeated until the centrifugate is practically clear, showing that all of the dispersible colloid has been removed.

The colloids are concentrated to a slurry by filtration with the candles. The slurry is dried at 105° C., powdered, and the powder stored in vials for analysis. The analyses are made by standard procedures for soil analysis.[4]

Base or ion exchange is defined as the substitution of a base for another base, or for hydrogen, in the soil. This action can best be illustrated by the description of a laboratory experiment. A sample of soil that showed an acid reaction was placed in a funnel and neutral potassium chloride was leached through the soil. The leachate was tested and found to be acid; on analysis the soil was found to contain more potassium than the original soil. The reaction can be written: Hydrogen clay + potassium chloride $\leftrightarrows$ potassium clay + hydrochloric acid. The process is reversible and follows well-defined chemical laws.

Base exchange determinations involve measuring (1) the base exchange capacity and (2) the kind and amount of the exchangeable bases. The method may be outlined as follows:

The portion of the soil sample passing the No. 40 sieve is leached with neutral ammonium-acetate solution. The leachate contains all the exchanged bases and the soil retains the absorbed ammonia which displaced the bases. The leachate is then analyzed for the amount and kind of bases exchanged. The soil is washed to remove the excess ammonium acetate and is then analyzed for the absorbed ammonia content. From the amount of ammonia absorbed, the base exchange capacity of the soil is calculated. The sum of the exchanged bases and the base exchange capacity are recorded in milliequivalents per 100 grams of sample taken from the roadway.

[3] The Fractionation, Composition, and Hypothetical Constitution of Certain Colloids Derived from the Great Soil Groups, by I. C. Brown and H. G. Byers, U. S. Department of Agriculture, Technical Bulletin No. 319, p. 8, 1932.

[4] Method and Procedure of Soil Analysis Used in the Division of Soil Chemistry and Physics, by W. O. Robinson, U. S. Department of Agriculture, Circular No. 139, 1939.

SILICA-SESQUIOXIDE RATIO INVESTIGATED

A study of the significance of the silica-sesquioxide ratio was made on 19 samples of soil-aggregate base courses taken from roads in service in Alabama and Georgia. The results of the physical tests on these samples and the condition of the road surfaces at the time the samples were taken are shown in table 1. Chemical analyses were made of the colloid fractions of these samples and the results are given in table 2, including the silica-sesquioxide ratios and the silica-alumina ratios.

These base courses had been surface treated with single or double applications of bituminous material and mineral aggregate. Since the resulting surfaces had a total thickness of only 1 inch or less, the condition of the surface was assumed to be directly dependent on the condition of the base course.

The Standard Specifications for Materials for Stabilized Base Course, M 56–38, of the A. A. S. H. O., require that the dust ratio, B/A (ratio of the fraction passing the No. 200 sieve to the fraction passing the No. 40 sieve) shall not exceed 0.50, and that the fraction passing the No. 40 sieve shall have a liquid limit not greater than 25 and a plasticity index not greater than 6.

Samples 2, 5, 11, 14, and 19 (table 1) met the requirements for dust ratio, liquid limit, and plasticity index of the A. A. S. H. O. specifications for base courses; and it is noted also, in these cases, that the roads were in satisfactory condition. Samples 1, 3, and 10 had higher plasticity indexes than the specifications permit but conformed in all other details. The roads from which these samples were taken were satisfactory.

TABLE 1.—*Physical test data of Alabama and Georgia base-course materials as related to road conditions*

Sample No.	Sieve analysis of samples, percentage passing—								Ratio B/A	Hydrometer analysis, percentage of total sample		Physical characteristics of material passing the No. 40 sieve						
	1½-inch sieve	1-inch sieve	¾-inch sieve	⅜-inch sieve	No. 4 sieve	No. 10 sieve	No. 40 sieve (A)	No. 200 sieve (B)		Silt, 0.05 to 0.005 mm.	Clay, smaller than 0.005 mm.	Liquid limit	Plasticity index	Shrinkage Limit (S)	Shrinkage Ratio (R)	Moisture equivalent Centrifuge	Moisture equivalent Field (FME)	Condition of road surface
1	--------	100	99	90	82	78	62	23	0.37	6	15	25	9	16	1.8	13	20	Satisfactory.
2	--------	--------	100	90	64	56	53	18	.34	4	11	18	5	13	1.9	9	14	Do.
3	--------	--------	100	98	96	83	48	19	.40	2	16	25	9	17	1.8	14	19	Do.
4	100	98	93	88	84	72	47	23	.49	3	19	42	16	18	1.7	20	23	Do.
5	--------	100	99	98	96	93	65	33	.51	13	18	19	4	13	2.0	10	15	Do.
6	93	90	88	85	81	74	50	31	.62	13	17	27	11	14	1.9	21	19	Fair.
7	--------	--------	100	99	97	94	70	45	.64	21	21	25	8	14	1.8	18	20	Do.
8	97	89	82	71	63	54	36	21	.58	11	9	44	21	18	1.7	23	32	Do.
9	94	88	78	61	53	46	32	19	.59	7	8	28	10	14	1.8	16	20	Do.
10	100	99	98	89	69	57	47	20	.42	5	12	24	10	14	1.8	13	18	Satisfactory.
11	--------	100	99	86	67	57	47	22	.47	7	10	18	4	14	1.9	12	14	Do.
12	--------	--------	--------	--------	--------	100	89	17	.19	2	15	32	10	23	1.5	12	30	Poor.
13	--------	--------	--------	--------	--------	100	94	21	.22	1	20	36	14	22	1.6	15	38	Do.
14	--------	--------	--------	--------	--------	100	74	20	.27	3	16	25	6	19	1.7	10	19	Satisfactory.
15	--------	--------	--------	--------	--------	100	59	21	.36	3	16	28	8	18	1.7	11	19	Do.
16	--------	--------	--------	--------	--------	100	61	20	.32	2	17	33	10	20	1.7	12	24	Do.
17	100	99	95	73	56	48	39	17	.44	5	12	28	12	16	1.8	18	20	Do.
18	--------	100	97	77	59	50	35	14	.40	4	11	31	11	16	1.8	14	24	Do
19	--------	100	95	80	65	58	45	15	.33	4	10	25	4	16	1.7	13	20	Do.

Samples 4, 12, 13, 15, 16, 17, and 18 had higher plasticity indexes and liquid limits than the A. A. S. H. O. specifications permit but the dust ratios were satisfactory. Of the seven roads of which these seven samples were representative, five, Nos. 4, 15, 16, 17, and 18, were in a satisfactory condition, and two, Nos. 12 and 13, were in poor condition. The remaining four samples, Nos. 6, 7, 8, and 9, had excessive dust ratios (*B*/*A* values, table 1), and high plasticity indexes and liquid limits. The road conditions in these four instances were reported as fair.

TABLE 2.—*Chemical analyses of colloids from Alabama and Georgia base courses*

Sample No.	SiO_2	Fe_2O_3	TiO_2	Al_2O_3	Loss on ignition	Molecular ratios	
						$\frac{SiO_2}{Al_2O_3+Fe_2O_3}$	$\frac{SiO_2}{Al_2O_3}$
	Percent	*Percent*	*Percent*	*Percent*	*Percent*		
2	35.0	8.3	4.4	35.2	17.4	1.5	1.7
5	31.7	12.7	2.4	35.8	16.7	1.2	1.5
11	32.3	11.7	3.9	35.7	17.7	1.3	1.5
14	30.1	25.7	2.7	28.3	13.5	1.1	1.8
19	42.2	11.4	1.8	31.5	12.4	1.9	2.3
1	37.7	12.5	4.6	31.7	14.5	1.6	2.0
3	34.8	14.2	3.4	32.3	14.6	1.4	1.8
10	33.6	10.9	4.9	34.5	17.1	1.4	1.7
4	34.9	15.2	4.1	32.2	14.4	1.4	1.8
12	42.9	10.3	2.8	30.0	12.8	2.0	2.4
13	42.6	10.3	2.7	31.2	12.8	1.9	2.3
15	35.6	17.7	2.1	30.3	13.3	1.5	2.0
16	35.6	19.7	2.9	28.6	12.8	1.5	2.1
17	39.3	15.4	2.3	29.5	12.4	1.7	2.3
18	38.8	15.8	2.2	29.3	12.5	1.7	2.3
6	39.2	12.0	2.1	31.8	13.9	1.7	2.1
7	41.2	9.9	1.7	32.5	13.7	1.8	2.2
8	41.5	8.4	2.2	34.2	14.6	1.8	2.1
9	42.8	8.2	2.6	33.7	14.6	1.9	2.2

The silica-sesquioxide ratio of soil colloids is controlled by the kind and amount of clay minerals present. The relation of the silica-sesquioxide ratio to the character and behavior of soils is hence a resultant of effect of the clay minerals. Soils with colloids having ratios higher than 2, indicating the presence of montmorillonite group clay minerals, tend to have greater volume changes than soils with ratios less than 2.

Values for the volume change and lineal shrinkage of the 19 base-course materials from Georgia and Alabama are given in table 3. Chemical analyses and silica-sesquioxide and silica-alumina ratios of six additional soils are given in table 4. Values for volume change and lineal shrinkage of these soils are given in table 5. Samples 38, 39, and 40 represent subgrade soils from Alabama and Georgia and samples 41, 42, and 43 represent soils from the vicinity of Washington, D. C. It will be noted that the silica-sesquioxide and silica-alumina ratios of the three northern soils are greater than those for the three southern soils.

In figure 1 are shown the relations between the lineal shrinkage and the silica-sesquioxide (SiO_2/R_2O_3)* and silica-alumina (SiO_2/Al_2O_3) ratios for samples 1 to 19 inclusive (tables 2 and 3) and samples 38 to 43 inclusive (tables 4 and 5). The curves of figure 1 are straight lines plotted by the method of least squares and show a definite trend toward an increase in lineal shrinkage with an increase in either the silica-sesquioxide or silica-alumina ratio. Physical test constants other than lineal shrinkage do not show this trend.

*$R_2O_3=Al_2O_3+Fe_2O_3$.

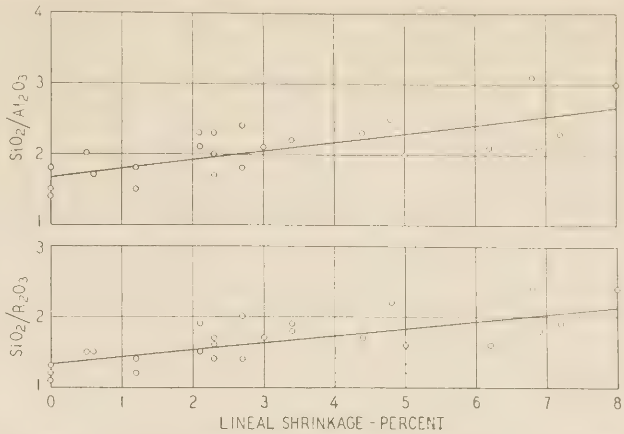

FIGURE 1.—THE RELATION BETWEEN SiO_2/R_2O_3, SiO_2/Al_2O_3 RATIOS AND LINEAL SHRINKAGE OF SOILS.

TABLE 3.—*Volume change and lineal shrinkage of Georgia and Alabama base-course materials*

Sample No.	Volume change $C_f=R\,(FME-S)$	Lineal shrinkage $100\left(1-\sqrt[3]{\frac{100}{C_f+100}}\right)$
2	1.9	0.6
5	4.0	1.2
11	0	0
14	0	0
19	6.8	2.1
1	7.2	2.3
3	3.6	1.2
10	7.2	2.3
4	8.5	2.7
12	8.5	2.7
13	25.6	7.2
15	1.7	.5
16	6.8	2.1
17	7.2	2.3
18	14.4	4.4
6	9.5	3.0
7	10.8	3.4
8	23.8	6.9
9	10.8	3.4

TABLE 4.—*Chemical analyses of colloid fraction of subgrade soils*

Sample No.	SiO_2	Fe_2O_3	TiO_2	Al_2O_3	Loss on ignition	Molecular ratios	
						$\frac{SiO_2}{Al_2O_3+Fe_2O_3}$	$\frac{SiO_2}{Al_2O_3}$
	Percent	*Percent*	*Percent*	*Percent*	*Percent*		
38	30.8	11.7	2.9	36.8	17.1	1.2	1.4
39	37.4	14.1	4.2	31.7	13.3	1.6	2.0
40	38.4	14.6	2.8	31.8	12.7	1.6	2.1
41	46.0	5.2	3.6	31.7	12.4	2.2	2.5
42	47.0	12.7	1.2	25.8	11.2	2.4	3.1
43	48.5	9.2	1.0	28.0	9.0	2.4	3.0

TABLE 5.—*Volume change and lineal shrinkage of subgrade soils*

Sample No.	Volume change $C_f=R\,(FME-S)$	Lineal shrinkage $100\left(1-\sqrt[3]{\frac{100}{C_f+100}}\right)$
38	0	0
39	16.5	5.0
40	21.0	6.2
41	16.2	4.8
42	23.4	6.8
43	28.5	8.0

Since there is a relationship between the silica-sesquioxide ratios and the physical properties of soils, there should be in turn a relationship between the service behavior and silica-sesquioxide ratio. That this is true is shown in table 6. Five base courses with soil aggregates conforming to A. A. S. H. O. specifications contained colloids with silica-sesquioxide ratios from 1.1 to 1.9. All of the roads represented by these samples gave satisfactory performances. Of the remaining 14 soil-aggregates which conformed only in part with the A. A. S. H. O. specifications, those with silica-sesquioxide ratios below 1.7 represent satisfactory roads, those having ratios of 1.7 or 1.8 are borderline cases varying between satisfactory and fair, and those with ratios of 1.9 and 2.0 are unsatisfactory.

TABLE 6.—*Comparison of condition of roads with chemical characteristics of colloids from base course materials from Alabama and Georgia*

Sample No.	Type of material	Condition of road	Silica-sesquioxide ratio
SOILS CONFORMING WITH A. A. S. H. O. SPECIFICATIONS (EXCEPTIONS NOTED)			
[1] 2	Georgia pebble soil	Satisfactory	1.5
5	Topsoil	do	1.2
11	Clay gravel	do	1.3
14	Sand clay	do	1.1
[1] 19	Clay gravel	do	1.9
SOILS WITH EXCESSIVE PLASTICITY INDEX			
1	Georgia pebble soil	Satisfactory	1.6
3	Sand clay	do	1.4
10	Clay gravel	do	1.4
SOILS WITH EXCESSIVE LIQUID LIMIT AND PLASTICITY INDEX			
4	Sand clay	Satisfactory	1.4
[1] 12	do	Poor	2.0
[1] 13	do	do	1.9
[1] 15	do	Satisfactory	1.5
[1] 16	do	do	1.5
17	Clay gravel	do	1.7
18	do	do	1.7
SOILS WITH EXCESSIVE DUST RATIO, LIQUID LIMIT, AND PLASTICITY INDEX			
6	Topsoil	Fair	1.7
7	do	do	1.8
8	Clay gravel	do	1.8
9	do	do	1.9

[1] Does not conform to A. A. S. H. O. gradation specification.

The base courses were from 5 to 8 inches thick so that it was not considered likely that the subgrade had a marked effect on the surface behavior of the roads represented by samples 1 to 19, tables 1, 2, and 3.

BASE EXCHANGE RELATED TO SURFACE COURSE PERFORMANCE

Since a relationship between the results of chemical and physical tests and the service behavior of soil-aggregates in base courses was shown, there should be a relationship between chemical factors and the behavior of soil-aggregates in surface courses. To test this assumption, a study of base-exchange characteristics was made on soil-aggregate samples from the surface courses of 14 roads in Maryland and 4 roads in Delaware.

Base exchange was considered to be of most significance in these surface courses because a large percentage of the surfaces tested had been treated with sodium chloride or calcium chloride. Base exchange measurements take into account the effect of the addition of these chemicals, whereas silica-sesquioxide measurements are independent of the addition of water-soluble chemicals.

The condition of these 18 soil-aggregate surface courses and the results of the physical tests on samples taken from them are given in table 7. Samples 20 to 23, inclusive, were taken from 4 road surfaces in Delaware that subsequently were to serve as base courses for bituminous surfaces. However, at the time they were sampled and their condition observed, they were being used as surface courses and therefore they are considered as such in this study.

The Standard Specifications for Materials for Stabilized Surface Course, M 61–38, of the A. A. S. H. O., require that the dust ratio, B/A, shall not exceed 2/3, and that the fraction passing the No. 40 sieve shall have a liquid limit not greater than 35 and a plasticity index not less than 4 and not greater than 9.

Samples 20 to 23, inclusive, failed to meet the specification requirements for surface courses in that their plasticity indexes were too low. However, the road conditions were satisfactory in the sections represented by samples 20, 21, and 22. The road surface represented by sample 23 was unconsolidated and very dusty in summer.

Materials represented by samples 24 to 37, inclusive, table 7, were taken from surface courses of Maryland roads. Sample 37 met the requirements of the A. A. S. H. O. specifications for surface courses, except for grading, but the road surface was soft and rutted under light loads. The plasticity indexes of samples 24, 30, 33, 34, 35, and 36, were low. The roads from which samples 24, 33, and 34 were taken were in good condition. The road represented by sample 30 became soft when wet and the surfaces of the sections represented by samples 35 and 36 were corrugated due to a deficiency of binder.

The section from which sample 25 was taken was in good condition and that represented by sample 26 was in faily good condition, although the dust ratios of these samples were higher and the plasticity indexes lower than the specifications permit. Sample 32, which had an excessively high plasticity index, was representative of a road having a number of pot holes (due in part perhaps to lack of maintenance). Sample 31, which had a high dust ratio, was taken from a road that was badly rutted and soft when wet. The remaining three samples, Nos. 27, 28, and 29, which had higher dust ratios, plasticity indexes, and liquid limits than the A. A. S. H. O. specifications permit, were taken from roads that were badly rutted.

Of the Delaware roads, the two represented by samples 20 and 21 gave the best service.[5] The third (No. 22) was satisfactory but not quite as good as the first two. The fourth sample (No. 23) represented material that was unsatisfactory. It did not consolidate and was very dusty in dry weather.

Base exchange data for the Delaware and Maryland surface courses are given in table 8. They show that, of the four samples from Delaware, sample 23 had a much lower base exchange capacity than the other three. These results indicate that the soil mortar of sample 23 contained a much less effective binder than the other three soils which show highly weathered and effective binders. Base exchange data and condition of the Delaware roads are compared in table 9.

[5] Reported by Maxwell P. Harrington, Soils Engineer, Delaware State Highway Department by letter—"When placed in the roadway they consolidate rapidly and become exceedingly hard. They are impervious to water when consolidated and do not become even slightly soft or muddy."

TABLE 7.—*Physical test data of Delaware and Maryland surface course material as related to road conditions*

Sample No.	Sieve analysis of samples, percentage passing—								Ratio B/A	Hydrometer analysis		Physical characteristics of material passing the No. 40 sieve						Condition of road surface
	1½-inch sieve	1-inch sieve	¾-inch sieve	⅜-inch sieve	No. 4 sieve	No. 10 sieve	No. 40 sieve (A)	No. 200 sieve (B)		Silt, 0.05 to 0.005 millimeter	Clay, smaller than 0.005 milli meter	Liquid limit	Plasticity index	Shrinkage: Limit	Shrinkage: Ratio	Moisture equivalent: Centrifuge	Moisture equivalent: Field	
20	------	------	100	98	94	94	55	13	0.24	5	8	21	1	23	1.6	14	22	Smooth and well compacted.
21	100	97	87	82	75	75	52	11	.21	3	8	19	0	21	1.7	13	21	Do.
22	------	------	100	99	98	98	68	13	.19	2	11	16	0	21	1.7	7	28	Do.
23	------	------	------	------	------	100	62	7	.11	3	4	15	0	26	1.6	3	22	Unconsolidated. Dusty in summer.
24	------	------	100	------	96	74	40	21	.53	10	9	18	2	------	------	13	------	Smooth and well compacted.
25	------	87	79	------	59	44	31	23	.74	13	9	25	3	------	------	19	------	Surface tight, but rough due to lack of maintenance.
26	------	------	100	------	88	62	40	28	.70	19	6	20	3	------	------	13	------	Surface has loose mulch on top and a few soft spots.
27	------	94	88	------	69	54	43	31	.72	16	13	41	10	------	------	34	------	Surface badly cut up and rutted.
28	------	97	90	------	63	52	39	31	.80	12	17	39	11	------	------	31	------	Rutted badly, soft when wet.
29	------	100	99	------	92	84	71	62	.87	34	25	39	10	------	------	32	------	Do.
30	------	------	100	------	69	43	23	13	.57	7	5	22	3	------	------	14	------	Surface smooth, soft when wet.
31	------	91	90	------	81	73	59	42	.71	21	17	34	8	------	------	26	------	Rutted, soft when wet.
32	------	95	94	------	62	48	35	16	.46	4	11	31	11	------	------	18	------	Pot holes, may be due in part to lack of maintenance.
33	------	98	95	------	67	54	29	12	.41	5	7	19	3	------	------	12	------	Satisfactory. Loose mulch on top.
34	------	100	99	------	82	66	43	12	.28	4	7	16	0	------	------	6	------	Smooth in good condition.
35	------	97	89	------	57	47	30	13	.43	2	6	17	0	------	------	8	------	Surface corduroyed. Not enough binder.
36	------	94	84	------	55	46	29	7	.24	2	5	15	0	------	------	5	------	Do.
37	------	100	99	------	88	77	55	28	.51	16	9	22	4	------	------	14	------	Surface soft, rutted.

TABLE 8.—*Base exchange data obtained with Delaware and Maryland soils*

Sample No.	Milli-equivalents [1] of base exchange capacity: Per 100 grams soil passing No. 40 sieve	Milli-equivalents [1] of base exchange capacity: Per 100 grams total sample	Predominant exchangeable base	pH
20	4.0	2.0	Ca and Mg	4.8
21	2.9	1.1	Ca	4.8
22	1.9	1.3	Ca	4.4
23	1.2	.7	Ca	5.0
24	3.3	1.3	Ca	7.6
25	6.7	2.1	Ca	7.8
26	2.3	.9	Ca	7.4
27	8.0	3.4	Ca	7.4
28	6.8	2.7	Ca	4.7
29	7.5	5.3	Ca	5.8
30	10.0	2.3	Ca	7.7
31	5.3	3.1	Ca	5.5
32	6.9	2.4	Ca	4.8
33	3.6	1.0	Ca	4.1
34	2.9	1.2	Ca	5.1
35	3.8	1.1	Ca	4.8
36	2.0	.6	Na and Ca	6.3
37	5.0	2.8	Ca	7.7

[1] $\text{Milli-equivalent} = 1000 \times \frac{\text{weight of substance in grams}}{\text{chemical equivalent weight of the substance}}$.

TABLE 9.—*Comparisons of road conditions with base exchange data of Delaware soils*

Sample No.	Type of material	Condition	Base exchange capacity milli-equivalents per 100 grams total sample
20	Clay gravel	Smooth and well compacted	2.0
21	do	do	1.1
22	do	do	1.3
23	do	Unconsolidated. Dusty in summer	.7

According to Hogentogler and Willis [6] the stability of a soil depends on the cohesiveness of the binder and the strength and permanency of the adhesion that can be developed between the coarser soil particles and binders. Therefore, if base exchange measurements indicate the effectiveness of the binder, the effectiveness in turn should also be indicated by physical tests if the diluting action of the sand can be eliminated. To test this assumption, the samples from Delaware roads were sieved through the No. 200 sieve and the physical tests made on the minus No. 200 material.

The test results, table 10, show that both the liquid limit and the colloid content of sample 23A are about half those of the other three. The low colloid content and low liquid limit are indicative of the low efficiency of the binder in the case of this soil-aggregate. Higher colloid contents in the other samples produced higher physical test constants and a more cohesive binder.

[6] Stabilized Soil Roads, by C. A. Hogentogler and E. A. Willis, PUBLIC ROADS, vol. 17, No. 3, May 1936, p. 45.

TABLE 10.—*Mechanical analysis and physical characteristics of Delaware soils passing the No. 200 sieve*

MECHANICAL ANALYSIS

Sample No.	Particles smaller than 2 millimeters (percentage by weight): Coarse sand, 2.0 to 0.25 millimeter	Fine sand, 0.25 to 0.05 millimeter	Silt, 0.05 to 0.005 millimeter	Clay, smaller than 0.005 millimeter	Colloids, smaller than 0.001 millimeter
20A	0	10	59	31	21
21A	0	5	49	46	28
22A	0	14	45	41	18
23A	0	12	39	49	11

PHYSICAL CHARACTERISTICS OF MATERIAL PASSING NO. 200 SIEVE

Sample No.	Liquid limit	Plasticity index	Shrinkage: Limit	Shrinkage: Ratio
20A	45	10	22	1.6
21A	44	14	20	1.7
22A	42	10	20	1.7
23A	21	0	20	1.7

For the 14 Maryland roads, samples 24 to 37, the condition of the roads and the base exchange characteristics of the corresponding soils are listed in table 11.

TABLE 11.—*Base exchange data and road conditions for Maryland soils conforming and not conforming with A. A. S. H. O. specifications*

Sample No.	Type of material	Conformity with specifications	Milliequivalents base exchange capacity per 100 grams total sample	Condition of road
[1] 37	Rock-soil	Conforms except for grading.	2.8	Surface soft, rutted.
24	Limestone screenings.	Plasticity index too low.	1.3	Smooth, well compacted.
30	do	do	2.3	Smooth, soft when wet.
33	Sand-gravel	do	1.0	Satisfactory, loose mulch on top.
34	Sand-clay-gravel	do	1.2	Smooth, in good condition.
35	Sand-gravel	do	1.1	Surface corduroyed, not enough binder.
36	do	do	[2] .6	Do.
[1] 25	Topsoil and limestone.	Plasticity index too low and dust ratio too high.	2.1	Surface tight.
[1] 26	Limestone screenings.	do	.9	Surface has loose mulch on top and soft spots.
32	Sand-gravel	Plasticity index too high.	2.4	Pot holes, may be due in part to lack of maintenance.
[1] 27	Gravelly	Liquid limit, plasticity index and dust ratio, all too high.	3.4	Surface cut up and rutted.
[1] 28	Stone fragments	do	2.7	Rutted, soft when wet.
[1] 29	do	do	5.3	Do.
[1] 31	do	Dust ratio too high	3.1	Do.

[1] Does not conform to grading specifications.
[2] Predominant exchangeable base in this soil is sodium. In all other soils calcium predominates.

Comparing the requirements of the A. A. S. H. O. specifications for surface courses with the physical test results and the conditions of the road surfaces, it may be noted that samples failing to meet the A. A. S. H. O. specifications due to a low plasticity index represented roads in both good and unsatisfactory condition. However, the base exchange capacity of the soil, calculated on the basis of 100 grams total sample, shows a striking correspondence with the service behavior of the road. Six roads (represented by samples 37, 32, 27, 28, 29, and 31) with soils of base exchange capacities ranging from 2.4 to 5.3 were in an unsatisfactory condition with surfaces badly cut up and rutted. The surface represented by sample 30, with a base exchange capacity of 2.3, was in good condition in dry weather but became soft in wet weather. Four samples (Nos. 33, 35, 36, and 26) having base exchange capacities of from 0.6 to 1.1 were from roads that were corduroyed or had loose mulch on top. The remaining three samples were all from roads in good condition, and for these samples the base exchange capacity varied from 1.2 to 2.1. Hence it is possible to correlate the service behavior of these roads with the base exchange capacity of the soil-aggregate in the road. The soils can be arranged into three groups according to their base exchange capacities as follows:

1. High base exchange capacity. The binder is overactive, causing detrimental effects resulting in roads with poor surfaces.
2. Medium base exchange capacity. This gives the desirable results of consolidation and bonding of the road surface.
3. Low base exchange capacity. The resulting detrimental effects are loose and unconsolidated material.

Soils can be classified in this manner even if they show uniform physical test constants because the chemical characteristics of soils seem to be the fundamental factors controlling physical properties. Particle size thus becomes a secondary factor. In some cases its effect is greatly modified by chemical variations.

Soils conforming to A. A. S. H. O. specifications are, in general, satisfactory. Those not conforming to the specifications may be satisfactory road materials because of their particular chemical properties. These chemical properties appear to be more important than has been generally realized.

Table 8 contains a column showing the results of hydrogen ion (pH) measurements on the Delaware and Maryland soils. They are included as a matter of record, but pH, in this study, has not been found to have any relation to either physical tests or service behavior.

SUMMARY

The data presented warrant the following conclusions:

1. Not all soils can be adequately classified for road-building purposes by physical tests alone. The chemistry of the soils must also be considered in many cases.
2. There is a general relationship between the silica-sesquioxide ratios of the colloid fractions of soils and the lineal shrinkages of the same soils.
3. The condition survey showed that roads having soil-aggregate base courses whose colloid fractions had low silica-sesquioxide ratios were in better condition than those with higher ratios.
4. Base exchange measurements indicate the activity of the binder portion of the soil.
5. The base exchange capacity of the binder soil was directly related to the service behavior of 18 road surfaces examined. Surface courses having base exchange capacities above 2.2 milliequivalents per 100 grams of sample were soft and rutted. With the exception of sample 21, those having base exchange capacities below 1.2 milliequivalents were corduroyed and did not consolidate. For values of the milliequivalent from 1.2 to 2.1, inclusive, the surfaces of the roads were in satisfactory condition.

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For the condition of ultimate failure in the stabilometer test, the value of C is taken as zero, and with reference to the computed values of C, table 2, it is seen that only for the condition of 100 percent maximum density is this soil able to deform in place without failure under a principal stress difference of 6,000 pounds per square foot which corresponds to a unit surface load of 10,344 pounds per square foot. For this particular table, a decrease of 4.8 percent in moisture content (24.5 to 19.7 percent) is attended by a relatively small gain in dry density (from 103 to 105 pounds per cubic foot). However, the increase in C corresponding to this moisture difference is quite large and is indicative of benefits derivable from having a well-compacted and stable subgrade.

TABLE 2.—*The effect of moisture content and compaction on the modulus, C, of a clay soil and on the settlement, S_L, of this soil when under a uniformly loaded circular area of radius 6.5 inches, $\mu=\frac{1}{2}$*

Soil moisture content	Dry density		Modulus, C, corresponding to principal stress difference, $v-l$				Settlement, S_L, corresponding to unit load, $p=\frac{v-l}{0.58}$, $\frac{z}{a}=0$			
			$v-l=500$	$v-l=1{,}000$	$v-l=3{,}000$	$v-l=6{,}000$	$p=862$	$p=1{,}724$	$p=5{,}172$	$p=10{,}344$
Percentage of dry weight	*Pounds per cubic foot*	*Percentage of maximum density*	*Pounds per square foot*	*Pounds per square foot*	*Pounds per square foot*	*Pounds per square foot*	*Inches*	*Inches*	*Inches*	*Inches*
35.2	86	82	50,000	[1] 0	[1] 0	[1] 0	0.17	(2)	(2)	(2)
30.5	92	88	170,000	80,000	[1] 0	[1] 0	.05	0.21	(2)	(2)
24.5	103	98	600,000	440,000	120,000	[1] 0	.01	.04	0.42	(2)
19.7	105	100	900,000	790,000	590,000	250,000	.01	.02	.09	0.40

[1] Samples failed at these principal stress differences.
[2] Failure.

CONCLUSIONS

On the basis of the earlier report [7] and on the computations and data in this report, it is believed that the following general conclusions are warranted.

1. For a uniform load on a circular area at the earth surface, failure of a cohesive supporting earth is most likely to begin at any point of a basin-shaped surface intersecting the axis of symmetry at a depth equal to about 0.7 of the radius of the loaded circular area and extending to the perimeter. For points on this surface, the principal stress differences have maximum values.

2. Since stresses of a certain magnitude may be insufficient to cause failure of the subgrade and yet sufficient to cause considerable settlement, depending on the properties of the subgrade, a knowledge of the stress-deformation characteristics of the subgrade soil is absolutely necessary.

3. It is believed that development of a method of estimating deflections of the subgrade on the basis of laboratory test data and without the necessity of loading tests in the field is a desirable objective.

4. A method is described for determining the modulus of deformation of cohesive soils by means of the triaxial compression device. In general this is a secant modulus that diminishes in magnitude as the principal stress difference is increased.

5. By substituting the appropriate value for this modulus in integrated expressions similar to those that apply to elastic behavior, it is possible to make some sort of estimate of the deflection of the supporting soil under load.

6. It is shown that the movement under stress diminishes according to determinable relations as the thickness of pavement is increased, assuming that both lateral yield and compaction within the flexible pavement itself is relatively small.

7. Values of settlement computed from theory and stabilometer test data indicate the possibility of very materially reducing deflections under wheel loads by having the subgrade compacted to maximum density.

[7] Stresses Under Circular Loaded Areas, by L. A. Palmer. Proceedings of the Highway Research Board, vol. 19, 1939.

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It would appear from the speed distributions that the effect of highway lighting at this hazardous location was a reduction in the average speed when the pavement was wet. However, concrete pavement has a tendency to appear slippery at night when wet, whether the light is from an overhead source or from the headlights of vehicles. Such accentuation of the appearance of slipperiness may account for this marked reduction in speed, but it is impossible from the results obtained to determine to what extent, if any, the overhead lights influenced vehicle speeds when the pavement was wet.

The speed counter was operated continuously for 96 hours at a position on the tangent at the top of the hill. This counter merely classified the vehicles by their speeds into 10 groups, and the total number of vehicles in each group was manually recorded at the end of each hour. Despite the fact that speeds at night under lights at this position may not be representative of normal behavior, this phase of the study is of particular interest because it represents perhaps the longest continuous record of vehicle speeds ever collected.

Table 4 shows the average speed of vehicles and the distribution of speeds in 10 groups. As might be expected, the average speed during daylight, 38.5 miles per hour, is faster than the average speed at night. The average speed with the lights on, 35.0 miles per

TABLE 4.—*Percentage of vehicles traveling in various speed groups during daytime and nighttime while pavement was dry and while pavement was wet* [1]

Speed group, miles per hour	Daylight hours, 7 a. m. to 5 p. m.		Night hours, 6 p. m. to 6 a. m.		
			Pavement dry		Pavement wet, lights on
	Dry pavement	Wet pavement	Lights off	Lights on	
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Below 19.1	2.8	3.6	2.6	5.2	2.4
19.1–25.0	3.2	5.6	4.6	6.4	4.5
25.1–29.7	7.5	10.0	9.8	11.1	11.8
29.8–35.1	19.1	22.4	24.6	25.4	29.5
35.2–41.4	30.1	32.7	31.4	25.9	31.9
41.5–45.4	17.0	13.6	14.0	13.6	11.5
45.5–50.3	12.6	8.0	7.6	8.4	5.8
50.4–56.3	5.7	3.1	3.4	2.8	2.1
56.4–59.5	1.1	.5	.9	.5	.2
Over 59.5	.9	.5	1.1	.7	.3
Average speed	38.5	36.5	36.8	35.0	35.8
Vehicles per hour	197	174	84	82	73
Number of vehicles studied	5,886	1,722	1,989	966	874

[1] Data for 8 hours of study between 6 and 7 a. m. and between 5 and 6 p. m. and for 1 hour at night with pavement wet, lights off, are not included in this table

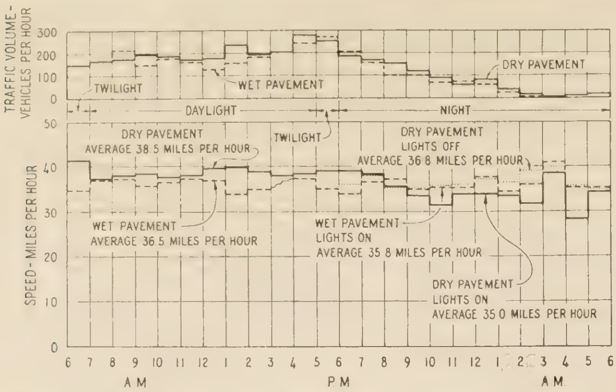

FIGURE 12.—TRAFFIC VOLUMES AND AVERAGE SPEEDS BY HOURS.

hour, is less than the average of 36.8 miles per hour found with the lights off. Wet pavement resulted in a decrease of speed in daytime, but a slight increase at night with the highway illuminated, a change that is inexplicable. It is significant that where a reduction of average speed occurs (table 4) it results from a general lowering of speeds in all speed ranges rather than because of a marked decrease in the number of vehicles in the higher speed groups. Under all conditions, some vehicles moved at 60 miles per hour or faster despite the 35-mile speed limit.

Figure 12 shows graphically the variation in average speed and traffic volume by hours. The consistency of the records is shown very clearly in this figure, for in no daylight hour was the average speed on wet pavement as great as that on dry pavement. This condition was reversed when the highway was lighted, as in the early evening, and thereafter throughout the night the average speed on wet pavement remained consistently higher than that when the pavement was dry.

SUMMARY

In interpreting the significance of the results of this investigation, several factors should be borne in mind.

1. The traffic volumes were relatively low, and the effect of lights on the speed of traffic as shown here may be entirely different from that for greater traffic volumes.

2. The alinement and grade of the short length of road studied may have prevented drivers from driving normally under any one condition.

3. The novelty of the lights being off for the first time in several years may well have had an influence on drivers accustomed to using the road.

4. The number of vehicles for which the speeds were recorded at night was not great.

5. The final criterion of effectiveness of lighting installations should be the effect on safety as revealed by before- and after-accident records.

These considerations almost preclude comparison of the results obtained in this study with those obtained elsewhere. Results of the study, however, seem to permit the following conclusions:

1. There are measureable differences in the behavior of drivers during daylight and darkness. These differences are most apparent in the transverse position of vehicles and in the frequency of passing. There is evidence that speed is also affected, but in this study the evidence cannot be considered conclusive.

2. The behavior of drivers operating on a lighted highway conforms very nearly to their behavior in daylight but does not conform to their behavior on unlighted highways at night, insofar as transverse position and passing frequency are concerned.

3. Conditions of illumination, as well as alinement and weather conditions, have no apparent effect on the normal distribution of time spacings between vehicles.

Other findings of general interest are:

4. A posted speed limit, when unenforced, has little effect upon the speeds of vehicles. Speeds as high as 82 miles per hour were recorded on the study section where the posted limit was 35 miles per hour, and hourly averages were seldom below this legal limit. This speed limit appears unreasonably low because the locality is distinctly rural in character although within corporate limits.

5. There is a certain minority of drivers on the road at all times who are prone to take risks. This is brought out rather clearly in the passing study, in which drivers were found passing where sight distances were entirely too short for safety.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF NOVEMBER 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 2,544,939	$ 1,265,304	73.6	$ 5,060,351	$ 2,516,990	196.0	$ 1,362,370	$ 678,110	47.6	$ 1,784,752
Arizona	1,028,980	681,907	47.7	1,308,250	922,445	60.8	423,225	233,341	9.6	777,956
Arkansas	4,047,747	1,890,989	107.8	1,345,803	667,127	56.0	468,153	233,770	18.5	246,174
California	4,689,174	2,439,857	100.2	8,470,058	4,436,259	114.8	2,109,617	1,089,870	50.4	809,415
Colorado	1,983,426	1,091,833	174.5	2,038,323	1,148,091	82.0	491,917	285,670	82.5	1,833,683
Connecticut	844,847	416,697	8.3	2,116,748	1,025,435	18.2	161,683	78,844	1.0	999,062
Delaware	275,496	137,748	12.0	1,917,176	931,150	20.6				1,040,813
Florida	1,253,119	626,559	33.3	2,757,533	1,368,488	83.6	1,548,161	743,451	26.0	1,996,268
Georgia	2,379,521	1,182,024	154.2	7,611,685	3,806,342	293.7	1,929,078	964,539	94.0	4,728,258
Idaho	1,356,626	826,505	146.4	1,148,359	702,959	60.5	104,377	64,380	5.7	1,327,372
Illinois	4,938,623	2,442,204	116.3	7,625,792	3,812,531	154.0	2,580,900	1,125,950	80.7	1,920,753
Indiana	2,668,729	1,331,306	63.4	7,799,859	3,749,773	135.0	2,529,760	1,232,755	38.1	511,267
Iowa	4,250,941	1,953,780	147.7	4,523,928	2,062,312	144.3	1,307,120	615,116	46.3	160,604
Kansas	3,312,848	1,634,983	259.1	5,998,208	3,017,866	367.6	2,792,849	1,396,439	178.9	2,728,642
Kentucky	1,829,848	910,529	47.4	3,574,038	1,787,019	96.2	1,318,356	659,178	60.0	2,472,207
Louisiana	902,986	445,939	15.9	12,238,713	3,168,308	54.3	1,389,039	686,831	37.8	2,878,612
Maine	1,311,088	649,700	28.6	718,171	359,085	21.1	41,700	32,600	.6	597,411
Maryland	786,000	390,500	18.9	3,354,558	1,671,362	32.5	507,303	253,651	5.9	1,183,412
Massachusetts	1,498,045	746,429	17.5	2,522,214	1,255,454	18.5				3,027,056
Michigan	3,151,089	1,509,738	117.0	10,423,610	5,111,205	274.7	902,863	396,432	33.7	335,045
Minnesota	3,628,763	1,758,810	328.5	6,481,807	3,234,231	361.3	1,381,445	690,105	38.2	2,606,685
Mississippi	1,518,991	580,278	54.6	6,943,874	3,226,931	351.3	1,450,160	702,330	90.8	1,291,597
Missouri	3,165,629	1,573,816	161.6	7,680,516	3,584,161	197.4	3,765,580	1,436,092	105.4	3,020,480
Montana	3,915,103	2,216,637	272.6	2,153,049	1,215,006	122.1	651,061	367,394	36.0	3,063,212
Nebraska	2,984,528	1,486,863	338.8	5,280,008	2,541,429	636.8	1,920,721	960,360	190.3	1,867,119
Nevada	1,404,181	1,208,112	74.6	1,177,081	1,025,149	48.7	209,733	182,842	16.3	573,042
New Hampshire	907,231	442,710	23.2	954,426	470,599	22.3				906,552
New Jersey	1,237,400	618,410	9.3	4,500,040	2,250,020	32.5	1,031,840	515,920	5.5	1,622,731
New Mexico	1,898,611	1,170,006	155.2	1,348,345	812,782	67.9	252,213	152,034	.4	970,165
New York	8,776,249	4,297,359	150.7	13,719,049	6,829,473	185.5	517,127	254,309	9.1	249,838
North Carolina	3,685,799	1,841,737	193.9	5,038,447	2,503,410	217.1	697,390	345,970	36.3	1,078,519
North Dakota	1,655,681	895,171	156.4	2,745,786	1,525,757	223.6	2,626,264	1,333,750	225.6	3,149,602
Ohio	2,518,056	1,258,803	35.9	13,274,952	6,613,147	119.5	6,125,468	3,037,254	48.1	2,568,555
Oklahoma	2,107,265	1,117,698	102.8	2,827,360	1,492,876	79.9	1,201,735	584,342	46.9	3,899,877
Oregon	2,991,449	1,788,011	148.4	1,867,670	1,090,876	36.7	1,191,489	569,740	30.3	564,360
Pennsylvania	3,829,928	1,887,523	54.6	13,121,040	6,509,866	119.4	3,782,878	1,886,212	23.3	893,215
Rhode Island	687,015	342,750	6.4	1,521,497	759,562	14.7				773,451
South Carolina	1,067,566	512,162	74.2	2,192,300	1,054,798	161.3	1,428,090	618,016	66.8	1,884,801
South Dakota	2,974,088	1,663,953	506.5	3,749,903	2,324,163	458.2	978,880	605,380	144.1	2,054,399
Tennessee	1,967,156	978,362	50.3	3,319,190	1,659,595	113.7	553,962	276,981	16.1	3,659,660
Texas	6,334,671	3,113,213	399.7	7,753,942	3,843,752	360.0	4,385,669	2,122,285	186.6	4,333,085
Utah	888,002	644,357	72.0	995,004	745,683	40.8	570,340	297,500	13.4	547,029
Vermont	1,163,682	581,760	36.6	621,508	308,948	15.9	378,009	189,004	8.7	200,755
Virginia	1,659,645	782,545	51.6	3,992,213	1,853,212	74.2	1,026,698	490,849	14.9	979,364
Washington	2,853,173	1,478,309	63.1	2,723,180	1,443,235	43.9	57,006	40,400	.3	672,307
West Virginia	1,465,280	729,175	44.6	3,051,584	1,519,591	85.3	546,060	273,030	7.6	1,494,543
Wisconsin	4,573,933	2,232,152	157.9	2,652,735	1,315,565	110.0	729,051	348,003	13.1	3,083,048
Wyoming	1,664,525	1,044,903	176.4	1,146,942	731,208	139.8				671,761
District of Columbia	345,488	172,744	2.7	363,742	173,837	2.8	539,519	238,200	.7	138,587
Hawaii	95,930	44,205	1.7	614,037	313,171	8.9	243,633	129,161	3.9	1,475,498
Puerto Rico	49,013	24,135	.5	1,675,709	829,135	27.7	196,527	96,495	3.0	621,005
TOTALS	119,068,103	61,061,200	5,595.1	218,040,313	107,321,369	6,763.6	60,407,019	29,514,885	2,199.0	82,273,604

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF NOVEMBER 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 190,944	$ 95,263	9.4	$ 1,303,357	$ 649,158	60.7	$ 5,000	$ 5,000		$ 276,238
Arizona	97,773	49,619	10.5	264,890	191,235	13.9				220,478
Arkansas	360,154	159,187	14.6	306,339	152,958	20.6	147,738	73,462	12.2	31,766
California	666,141	362,320	30.3	498,100	270,463	12.3	176,865	92,100	5.0	701,337
Colorado				264,057	148,305	5.8	39,261	22,127	1.2	119,790
Connecticut	287,102	139,108	3.3	188,388	90,213	3.1				165,242
Delaware	69,537	34,768	7.8	104,843	44,274	7.7				265,201
Florida	12,030	6,015		649,252	299,153	14.7	550,715	241,531	5.8	81,912
Georgia	79,007	38,969	10.1	738,149	354,074	62.2	441,783	220,892	43.5	1,000,215
Idaho	111,793	68,954	20.4	193,866	116,204	10.1	87,650	13,350	3.7	105,338
Illinois	1,489,074	737,507	77.5	1,101,200	535,600	29.5	293,600	130,500	16.4	127,321
Indiana	458,671	224,658	31.0	126,882	63,441	5.5	173,600	86,800	10.4	858,468
Iowa	2,017,784	965,451	430.3	808,570	383,540	196.9	329,734	143,693	110.1	53,528
Kansas	268,725	134,166	47.4	918,594	462,205	44.0	216,986	108,493	17.0	1,130,938
Kentucky	747,573	248,935	61.5	398,932	114,095	14.4	499,151	125,751	21.3	254,327
Louisiana	67,133	33,567	8.1	230,796	115,343	17.6				460,035
Maine	287,862	137,140	16.5	51,596	25,798	2.0				5,551
Maryland	94,300	47,150	3.5	87,390	43,695	4.7	86,000	42,500	6.1	383,032
Massachusetts	287,314	142,027	6.0	350,681	174,338	7.5	37,950	18,975	.4	476,543
Michigan	1,046,399	514,992	76.6	733,040	366,520	64.3	470,120	235,060	38.7	351,943
Minnesota	438,119	214,615	52.7	927,189	463,594	150.8	311,889	155,945	37.7	763,867
Mississippi	172,962	86,481	10.6	717,352	353,176	39.0	366,200	160,615	14.3	435,223
Missouri	630,479	314,837	79.1	193,532	96,766	24.3	341,368	135,145	36.3	614,885
Montana	641,506	362,577	80.3	82,618	46,621	3.7	202,812	111,045	44.1	536,900
Nebraska	360,685	181,971	65.6	661,481	322,458	87.3	163,632	81,816	28.8	205,900
Nevada	176,020	147,191	37.1	136,697	118,675	11.4	63,619	55,463	6.8	12,457
New Hampshire	143,639	68,883	3.4	2,192	1,096		83,441	40,100	3.8	127,711
New Jersey	319,500	159,750	10.6	318,057	158,940	11.4				503,134
New Mexico	101,564	59,154	13.1	634,137	343,277	28.8				91,225
New York	1,518,945	719,923	50.3	1,852,301	926,151	57.6				64,956
North Carolina	637,905	317,844	60.9	535,663	269,773	46.8	155,240	72,820	14.6	218,163
North Dakota	42,880	24,583	.3	169,224	90,702	3.6				1,014,689
Ohio	957,917	478,893	33.7	2,370,790	1,184,135	76.2	294,520	147,260	13.4	677,442
Oklahoma	624,887	331,437	43.8	244,665	128,986	14.6	172,130	90,442	12.8	849,896
Oregon	372,237	205,770	56.4	205,058	77,934	13.4	182,677	88,260	18.2	168,370
Pennsylvania	1,198,605	591,779	45.8	1,125,327	561,755	24.1	255,492	127,746	5.0	147,841
Rhode Island	157,358	78,624	3.4	89,358	44,679	.2	84,274	42,080	.9	44,667
South Carolina	211,510	85,994	30.0	552,493	209,873	62.8	504,867	185,200	48.7	72,895
South Dakota				28,926	19,392	9.0				1,265,145
Tennessee	139,047	67,467	10.9	193,926	96,963	7.2	101,734	50,867	2.8	865,132
Texas	1,161,661	568,592	168.8	952,228	460,613	76.0	500,200	242,122	51.5	568,720
Utah	54,999	34,100	9.2	215,530	138,660	22.4	61,685	14,729	2.5	110,203
Vermont	294,751	98,834	11.3	247,776	71,330	9.4				4,391
Virginia	330,770	153,923	20.0	408,412	185,070	15.3	204,652	101,655	9.3	184,715
Washington	476,066	250,888	28.2	192,524	103,039	3.9	235,379	125,900	24.5	114,034
West Virginia	301,750	150,350	16.7	35,469	17,734	1.8	127,225	63,612	4.5	411,469
Wisconsin	232,565	115,254	3.3	698,927	349,360	26.6	202,796	100,130	7.8	555,947
Wyoming	433,925	253,344	42.8	146,623	90,685	9.1				75,261
District of Columbia	61,300	30,650	.7	69,384	34,192	.7				22,703
Hawaii	275,662	137,588	8.6	1,096	1,096					159,971
Puerto Rico				302,225	147,640	14.0	55,188	27,140	2.1	80,408
TOTALS	21,108,530	10,431,092	1,862.4	23,630,102	11,714,977	1,448.9	8,227,173	3,780,326	682.2	18,037,523

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1939. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP . . Roadside Improvement. 10 cents.

No. 272MP . . Construction of Private Driveways. 10 cents.

No. 279MP . . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

Highways of History. 25 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP. . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

Indexes to PUBLIC ROADS, volumes 6–8 and 10–19, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF NOVEMBER 30, 1940

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	NUMBER: Grade Crossing Structures Reconstructed	NUMBER: Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	NUMBER: Grade Crossing Structures Reconstructed	NUMBER: Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	NUMBER: Grade Crossings Eliminated by Separation or Relocation	NUMBER: Grade Crossing Structures Reconstructed	NUMBER: Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 21,828	$ 21,828			2	$ 723,776	$ 703,700	7		2	$ 26,439	$ 26,439		1	5	$ 916,112
Arizona	192,342	184,976	3			179,037	178,688	1			19,260	19,260			4	232,120
Arkansas	357,939	357,794	3	1	7	973,893	969,672	10			144,553	144,553	1		9	332,818
California	439,428	439,428	4		6	635,043	456,561	1	1		422,200	420,083	6		13	1,323,225
Colorado						288,868	288,868	1			3,401	3,401			1	915,509
Connecticut	270,224	264,463	1	1		517,556	512,317	6								449,334
Delaware	65,760	65,760			12	124,808	124,808	1		4	13,839	13,839	4		1	460,294
Florida	207,524	203,025	2		2	102,816	102,291	1	1		139,739	139,739			25	1,209,730
Georgia	166,493	166,398	4		12	1,031,420	1,031,420	10	6		317,285	317,285	1	3	6	1,748,913
Idaho	212,890	209,461	5		5	31,172	31,172			7	62,584	62,584			23	411,509
Illinois	637,771	573,151	4	1	32	2,315,032	2,099,250	9	1	66	172,493	158,668			55	1,963,597
Indiana	479,293	479,293	3	1	44	702,019	702,019	4	1	42	45,928	45,928			14	995,474
Iowa	409,261	349,623	4		60	212,449	180,733	2		26	187,279	185,287	2		18	1,069,359
Kansas	679,019	678,912	9		12	301,380	300,902	3	1	4	404,040	404,040	7	1	11	869,617
Kentucky	170,478	169,935	4		11	1,037,019	1,037,019	11			39,110	39,110	1		4	545,571
Louisiana	95,496	95,496	1			345,122	291,627	2			630,384	572,654	11		1	802,109
Maine	159,759	158,841	1	1	6	19,574	19,574		1		125,990	125,990	1			251,016
Maryland	180,997	180,993	1		2	476,609	444,816	2	2	5	15,600	15,600			3	767,315
Massachusetts	15,710	15,710				343,592	333,170	1	1							2,018,040
Michigan	991,140	957,826	7	1	20	1,504,623	1,504,623	3	4	9	77,972	77,972		1	12	639,067
Minnesota	777,244	769,387	8	1	8	1,333,117	1,333,117	9	5	10	124,232	124,232		1	2	1,009,373
Mississippi	68,960	68,960	1			604,334	604,334	9	1		253,300	253,300	2		4	503,812
Missouri	779,990	779,991	3	5	1	2,138,903	1,683,483	8	1		62,344	30,678		1		1,006,711
Montana	427,675	427,675	5		2	82,097	82,096	1			9,155	9,155			3	350,584
Nebraska	224,026	222,355		1	7	821,838	821,838	12	1	3	238,849	238,849	4		10	241,722
Nevada	14,934	14,934			13	125,527	125,527	2		6	75,953	75,953	1		6	56,527
New Hampshire	100,989	100,953	3			149,458	148,638	3		1						325,194
New Jersey	269,185	269,185	2			595,071	595,071	3	1	3	266,607	266,607	1		1	924,243
New Mexico	140,504	140,504	1		6	304,092	274,896	4								482,206
New York	1,030,420	1,011,325	7	3		3,184,529	3,118,563	5	17		244,245	240,245		1	8	2,864,258
North Carolina	472,772	472,709	7	2	13	707,589	707,349	7	4	7	167,200	167,200		2	28	725,619
North Dakota	461,480	461,295	5		19	396,280	396,280	4		4						529,158
Ohio	630,858	609,096	5		14	2,203,465	2,154,275	9	3	2	961,257	933,494	6		1	2,089,292
Oklahoma	260,776	259,749	2	1	26	673,757	668,994	11		3	118,180	118,180			40	1,883,430
Oregon	208,639	117,537	3			254,208	197,981	1			5,790	5,790			2	357,935
Pennsylvania	943,046	934,003	12	1		1,990,424	1,986,163	12	1		1,132,399	1,132,399	8			2,836,327
Rhode Island	3,831	3,750				192,501	192,501		1							94,451
South Carolina	362,765	362,765	4	3	8	150,518	150,115	1		12	255,617	255,017	2		32	820,477
South Dakota	129,470	129,470	2	1	4	583,109	582,249	14			101,060	85,110	2			744,090
Tennessee	204,265	201,038	1	2		173,264	173,264	1	1		33,594	33,594		1		1,720,327
Texas	1,268,167	1,263,359	11		8	1,131,928	1,121,563	10	1	1	218,610	202,630	4		2	1,805,573
Utah	20,001	19,790			5	43,620	42,878	2		10	113,915	113,915			46	229,303
Vermont	97,595	97,595		1	2	151,541	151,541		1	7	8,182	8,182			3	213,187
Virginia	90,201	89,865	1	2	5	687,479	686,149	6		1	135,932	135,932		2	9	593,616
Washington	238,261	238,261	3	1	5	235,287	233,787	2	2	1	419,115	411,513	2	2	6	72,446
West Virginia	5,400	5,400			2	229,982	229,982	2			70,190	68,100	2		1	1,076,437
Wisconsin	814,012	799,461	6	3	7	473,301	444,271	3	1	4	114,346	104,781	1		10	1,274,601
Wyoming						562,886	562,885	6			4,997	4,997			2	166,634
District of Columbia						59,061	59,061	1								176,925
Hawaii						198,846	198,839	2		1	4,684	4,684			1	292,509
Puerto Rico						584,007	579,336	11								414,857
TOTALS	15,798,818	15,443,325	148	33	388	32,887,827	31,620,256	236	60	241	7,987,849	7,796,969	69	16	422	43,802,553