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Public Roads

A JOURNAL OF HIGHWAY RESEARCH

The Hybla Valley tests will add much to our knowledge of bituminous pavement performance

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GENERAL SERVICES ADMINISTRATION

E. A. STROMBERG, Editor

A Cooperative Study of Structural Design of Nonrigid Pavements

Second Progress Report

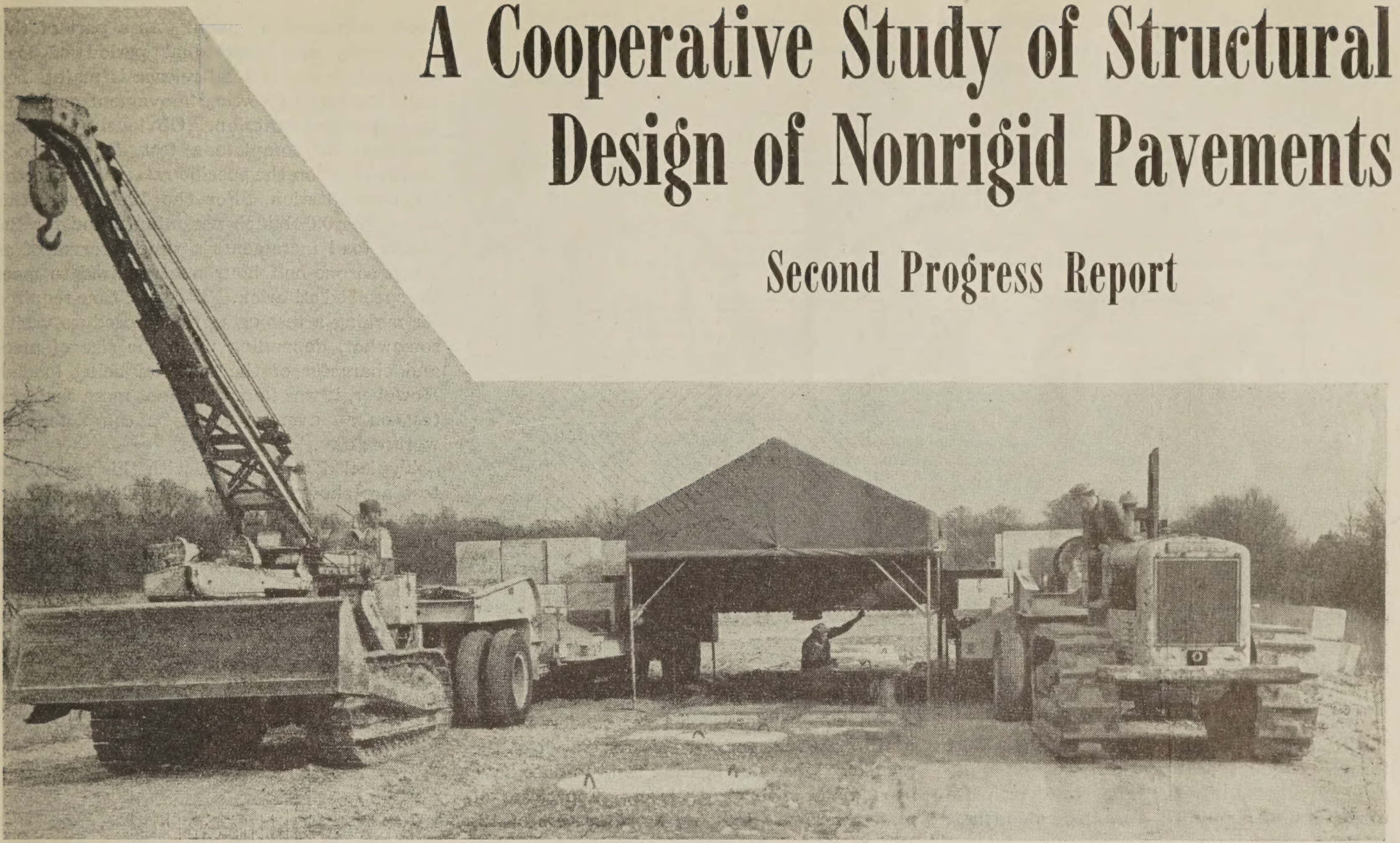


Figure 1.—The load reaction trailers

A DESCRIPTION OF FURTHER DEVELOPMENTS IN TEST METHODS AND INSTRUMENTATION

Reported by R. J. LANCASTER, Highway Research Engineer, Bureau of Public Roads and J. E. DRISCOLL, Research Engineer, the Asphalt Institute

This article is a second progress report on the cooperative investigation of the structural design of nonrigid pavements. The extensive experiments will result in the collection of a large body of important data and, it is hoped, in the development of significant conclusions of great value.

The principal objectives of the investigation include development of the load-supporting values of nonrigid pavements by full-scale field tests and correlation of these data with laboratory tests and with in-place determinations of various values of the base-course and subgrade components.

The first progress report provided a description of the investigation, its purposes, and proposed methods of procedure. This second progress report describes some innovations in plate-load testing techniques and equipment, including a relatively rapid method of testing which gives both complete load-deflection relations and information pertaining to the behavior of the components of a nonrigid pavement.

THE introductory report of this study was presented in PUBLIC ROADS over a year ago.[1] In addition to outlining the objectives of the study it included a descriptive account of the construction of the test pavement and a discussion dealing with the development of test apparatus and testing techniques.

The accumulation of test data from the investigation has not progressed to a point that warrants presentation of the findings at this time.[2] However, it is felt that a description of further developments in test methods and instrumentation might currently prove of interest and value to others carrying on research work in the same field. This report, then, consists principally of a detailed description of the methods of load testing that have been used in the study to date.

The study is a cooperative undertaking of the Highway Research Board, the Asphalt Institute, and the Bureau of Public Roads It is being conducted on an oval test track having two parallel tangents 800 feet in length connected at each end by a circular curve of 200-foot radius. The site of the experiment is near Hybla Valley, Va., 10 miles south of Washington, D. C.

The salient features of the pavement sections being tested at the present time are shown in figure 2. They consist of four main sections, each 200 feet in length and 44 feet in width, having base-course and surface-course thicknesses as indicated. The sections of pavement were built upon an embankment of uniform A–6 soil. The stabilized-aggregate base mixture was designed to conform with specification M 56–42, Type B–1, of the American Association of State Highway Officials, modified to permit the use of a slight amount of oversize material. The bituminous concrete was of the hot plant-mixed type, conforming to the Asphalt Institute dense graded aggregate specification A–2–a. Mix No. IV of this specification was used for all 3-inch surfacing and for the top 3 inches of

[1] *A cooperative study of structural design of nonrigid pavements*, by A. C. Benkelman and F. R. Olmstead, PUBLIC ROADS, vol. 25, No. 2, December 1947.

[2] This paper was presented at the 28th annual meeting of the Highway Research Board, December 1948.

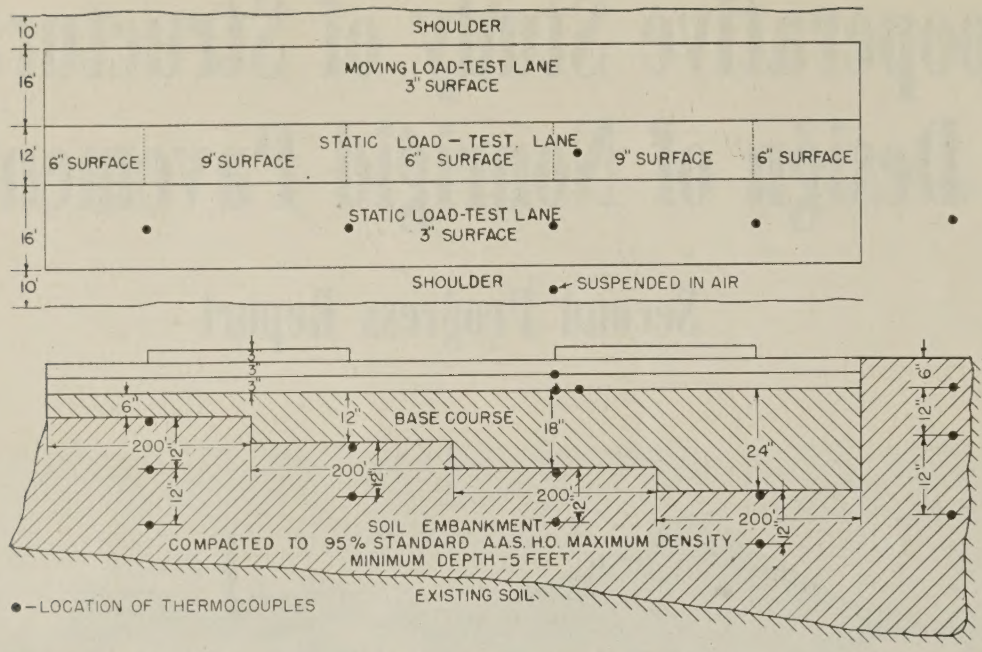


Figure 2.—Plan and profile of pavement sections, and locations of thermocouples.

the 6- and 9-inch surfacing. Mix No. II was used for the remaining lower portions of the 6- and 9-inch surfacing.

ORIGINAL TESTING PLANS

The original plans of the investigation called for the conduct of tests of both the static and moving-load type. The static or plate tests, totaling about 500, were to be made first. It was realized that the execution of these tests would require a very long period of time, particularly if but one test could be made per working day and if, because of climatic conditions, testing had to be restricted to a certain season of the year. A series of exploratory tests had demonstrated clearly that consistent results could not be anticipated if tests on the asphaltic concrete were conducted at pavement temperatures appreciably below 80° or above 90° F. In the region of Washington, D. C., pavement temperatures of this range in an area shaded from the sun prevail generally during the four summer months only, or for a period of about 80 working days.

In the early stages of the investigation consideration was given to the development and possible use of some type of load-bearing surface that would more closely simulate the effect of loads applied through tires than could be obtained with rigid plates. Several types of bearing surfaces were developed that gave promise of accomplishing the desired result. However, the use of another innovation of testing (simultaneous deflection measurements) which will be described later rendered their use impractical and for this reason all of the regularly scheduled bearing tests to date have been made with rigid plates. Four sizes of plates were selected, having diameters of 12, 18, 24, and 30 inches respectively. Two 25-ton capacity low-bed trailers were procured which, when loaded with concrete blocks and cross-connected with a heavy reaction girder (see fig. 1) provided a fairly mobile reaction load up to the 110,000-pound maximum capacity of the available hydraulic jacking system.

INCREMENTAL-REPETITIONAL LOADING TESTS

During the summer of 1947 about 60 plate-load tests were made, using an incremental-repetitional procedure. This procedure, which was mentioned in the first progress report, provided for the application of four load increments, each increment being applied and released five times. The magnitude of each load increment is such as to produce an increment of deflection of 0.125 inch. Thus the gross deflection of the material being tested will total 0.5 inch. As a part of this procedure, each individual period of load application and of load release is maintained until the rate of vertical movement conforms to a specified criterion. Obviously the time necessary to complete a test depends to a great extent on the specific rate value selected for this criterion. For the 1947 tests, this value was 0.001 inch per minute and for the higher load increments a time interval of as much as one-half hour was required to meet the specified criterion. The total time required for making a test using this procedure varied somewhat, depending upon the size of plate and character of the material being loaded. However, it was found that not more than one test on an average could be completed per working day.

Typical data obtained with this method of test are shown in figure 3. This is a load-deformation plot of the surface of the pavement only. Comparable data were obtained simultaneously at the surfaces of the base course and of the subgrade, so that two additional graphs of this nature are required to show all the data resulting from one test.

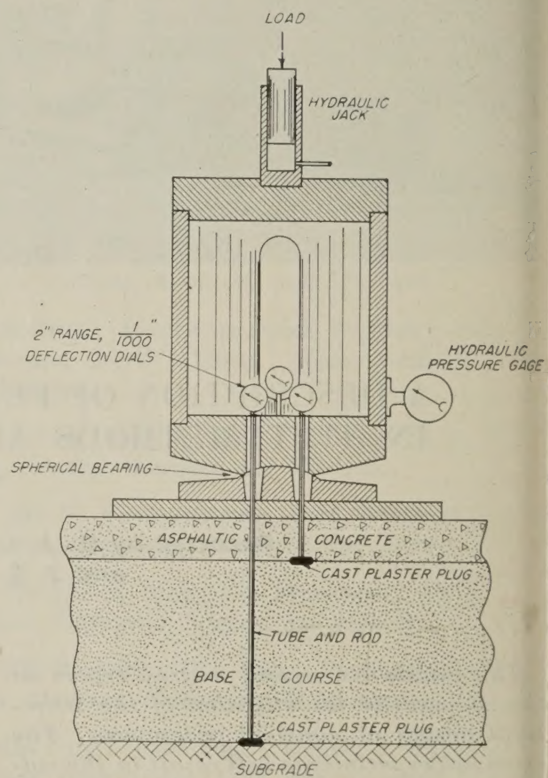


Figure 4.—Method of measuring simultaneous deflections at three levels of the pavement structure.

ACCELERATED LOAD-TEST PROCEDURE

Mention was made in the first progres report of a rapid method of testing that wa the subject of some experimentation durin preliminary stages of the investigation.

The outstanding feature of this so-calle accelerated method of test involved the con tinuous application of load until the resistanc of the material was overcome. The rate o application of the load was controlled so as t produce a constant rate of deflection of th material under test of 1.2 inches per minute This meant that with an ultimate or end-poin deflection of 2 inches, the actual time involve

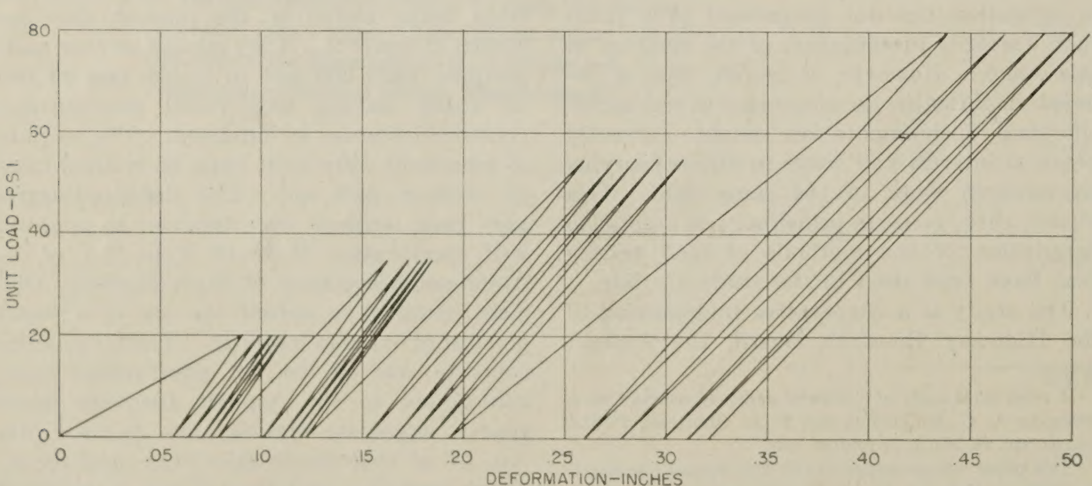


Figure 3.—Typical load-deflection and load-settlement data: 1947 procedure.

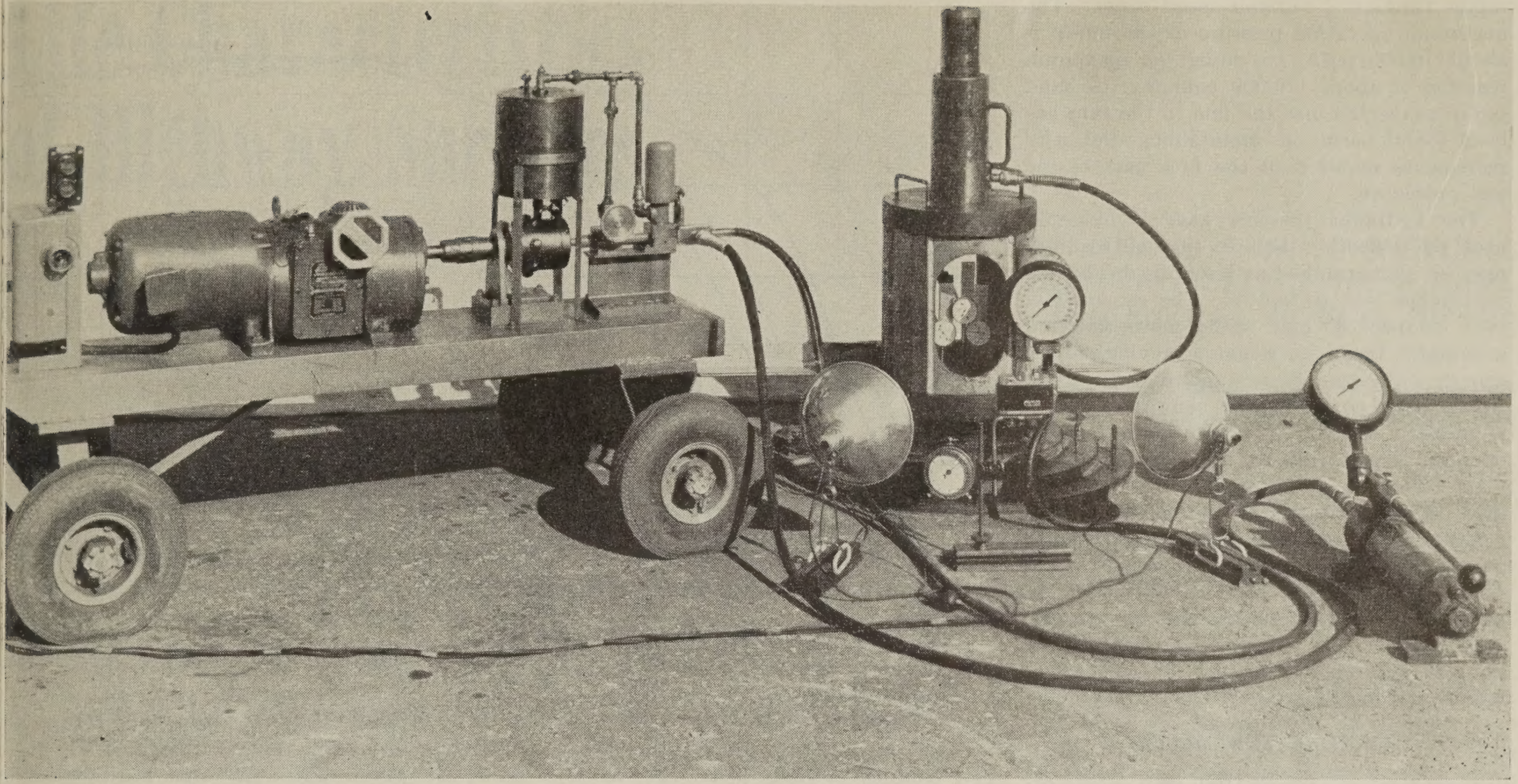


Figure 5.—Typical load-test set-up.

in making a test was less than 2 minutes, and that with adequate personnel and equipment a very large number of tests could be made in one day.

The results of the preliminary work using the accelerated procedure of loading were unusually well defined with respect to the effect of the size of loaded area and thickness of the pavement, the two most important variables being studied. Also, as a general rule, excellent agreement was found to exist between the results of duplicate tests. Moreover, with this testing procedure ultimate load values are obtained—information which it is believed will prove helpful in the analysis and correlation of the pavement load-test data.

The points enumerated above all entered into the decision to conduct a comprehensive series of tests utilizing the principal features of the accelerated test. However, the procedure as outlined above was modified to include the application and release of three load increments prior to applying the load continuously. This modification was introduced in order to obtain some information on the elastic action of the pavement components at loads less than ultimate.

STEPS IN PROCEDURE

The procedure, as modified and adopted for the 1948 series of tests, consists of the following steps:

1. Application of load sufficient to produce a deflection of approximately 0.2 inch.
2. Maintenance of the load until the rate of deflection slows to 0.001 inch in 15 seconds.
3. Release of the load until the rebound slows to 0.001 inch in 15 seconds.
4. Repeat steps 1 to 3, with deflection of of 0.3 inch in step 1.
5. Repeat steps 1 to 3, with deflection of 0.4 inch in step 1.
6. Continuous application of load so as to produce a rate of deflection of 0.5 inch per minute until a deflection of 2 inches is reached.
7. At a location just outside the influence of the test load the moisture content and density of the base course are determined, an undisturbed core of the subgrade is taken for a triaxial test, and moisture samples of the subgrade are taken from depths ranging up to 3 feet.

The word "deflection" as used above refers to the downward movement of the bearing plate, under load, from its initial position at the start of the test. In addition to measuring the deflection of the surface of the pavement, a method was developed by means of which the deflection at the top of the base course and at the top of the subgrade could be recorded simultaneously. This is accomplished by measurements made with vertical rods which pass through two holes, each 1½ inches from the center of the bearing plates, and into matching holes drilled to the proper depths in the pavement structure (see fig. 4). To eliminate friction between the vertical rods and the materials through which they pass and to provide some lateral support for these materials, metal tubes or casings are inserted in the drilled holes. Care is taken to provide clearance between the lower end of the tube and the seat of the deflection rod in order that any compression of the upper layers which might develop during the test would not bring the tubes in contact with the deflection rod seats. These seats consist of a hardened plaster plug or bulb formed by extruding soft plaster through the tubes by means of a snugly fitting plunger. This method was developed to overcome the tendency of the material at the base of the holes, particularly the subgrade, to intrude into the openings during the test.

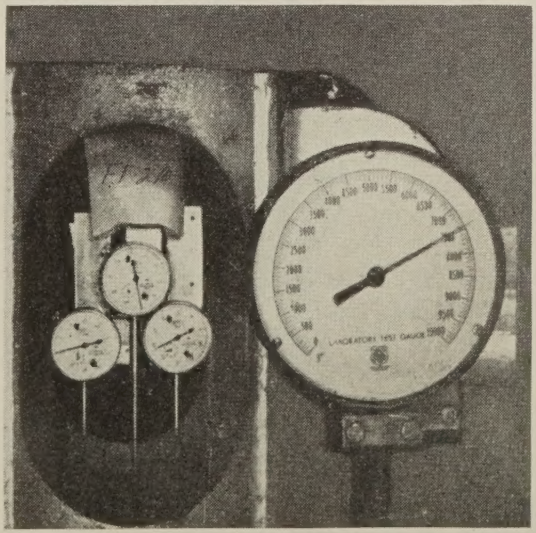


Figure 6.—Typical photographic record of deflection dials and load-pressure gage.

A photograph of a typical load set-up with the 30-inch diameter bearing plate is shown in figure 5. The 24-, 18-, and 12-inch plates are pyramided on the 30-inch plate for rigidity. Above the bearing plates is a spherical bearing surface. Resting on this surface is a cut-out cylindrical section in which the special 2-inch range-deflection measuring dials are suspended by arms extending from the deflection or dial-support beam. The cover at the upper end of the cylindrical section supports a 50-ton hydraulic ram. Supplying oil to the ram is a high-pressure, low-volume, radial-type hydraulic pump driven by an electric motor through a variable-speed drive. The maximum operating pressure of the pump is about 10,000 p.s.i., providing the maximum reaction of about 110,000 pounds. A hand pump connected into the line to the ram has been found useful in maintaining the load increments required in the first part of the test procedure.

The hydraulic pressure gage is mounted near the deflection dials so that all readings may be photographed at intervals during the continuous test, to failure. The camera is a very compact 35 mm. spring-motor-powered automatic type. It is not a moving-picture camera, but takes a picture and rolls up the film ready for the next exposure each time a button is pressed. During the progress of a test the operator takes a picture at each 0.1 inch of plate deflection as indicated by the measuring dial. After the film has been developed the negative is projected with a strip-film projector on a small screen from which readings of the simultaneous values of load, deflection of pavement surface, deflection of base course, and deflection of subgrade are taken. A sample frame taken from a film record is reproduced in figure 6.

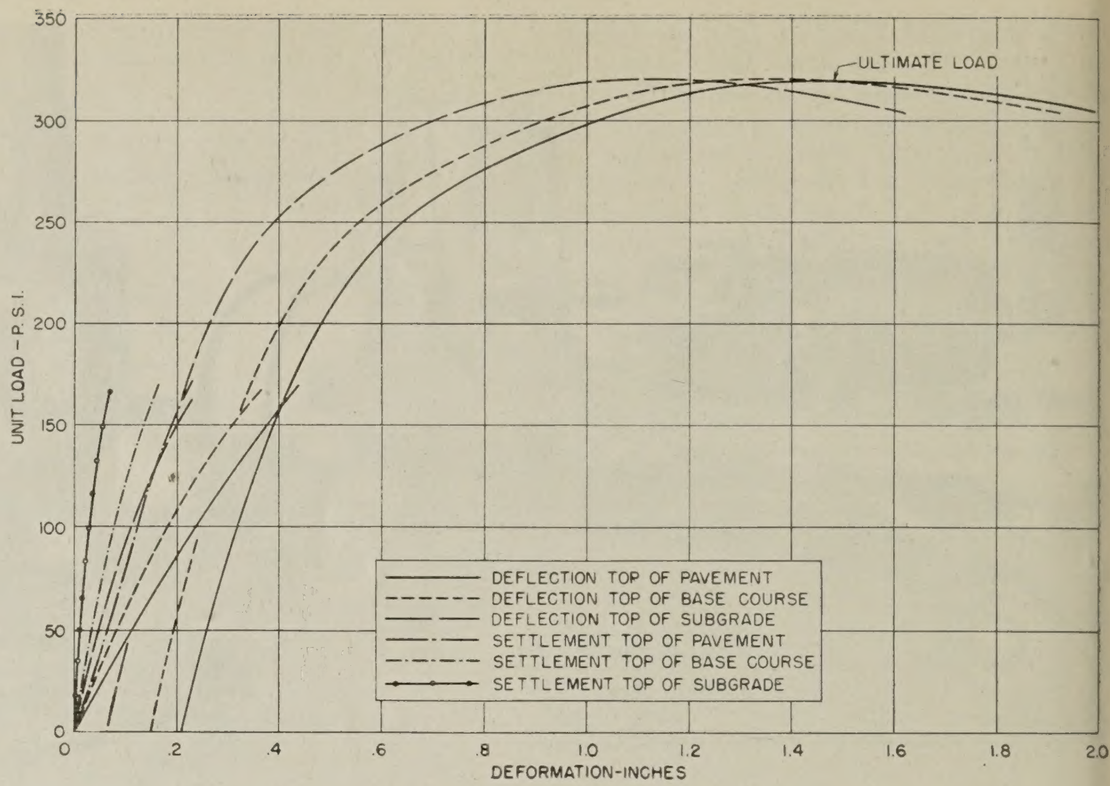


Figure 7.—Typical load-deflection and load-settlement data: 1948 procedure.

TYPICAL TEST DATA

Typical data resulting from this test procedure are shown in figure 7. The shorter lines radiating from the origin represent the first part of the test in which the load is applied and released in three increments. Three of these lines are the load-deflection relations for the three components of pavement structure; that is, the surfacing, base, and subgrade. The other three are the load-settlement relations of the same three components of pavement structure. The remaining three curves that sweep across the graph are the result of the latter part of the test, in which the load is applied continuously until a deflection of 2.0 inches is reached. These are load-deflection relationships starting from the last settlement values obtained for the surface, base, and subgrade, respectively. It is to be noted that an ultimate load value is reached, after which the curves descend slightly. As stated before, in this type of test the load is applied in such manner as to give a constant deflection rate of 0.5 inch per minute.

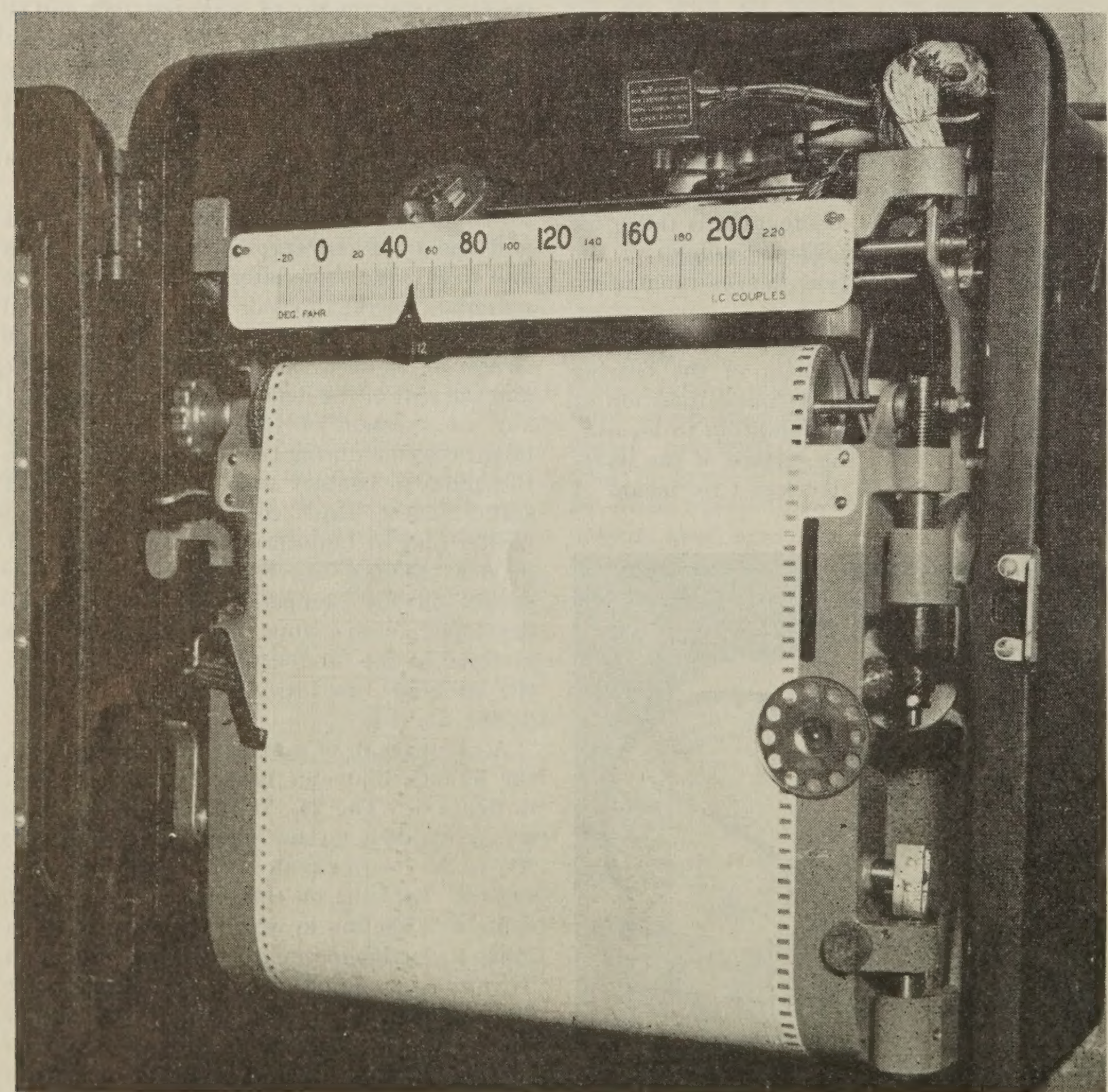


Figure 8.—Thermocouple temperature recorder.

In addition to load testing the surface of the pavement, tests are also made directly on the base, with the surfacing removed, and on the subgrade with the total depth of pavement removed. All tests are made in duplicate.

Tests on the base course and subgrade are made in two ways: one, with surcharge, where the opening in the overlying pavement components is of a size just sufficient to accommodate the plate; the other, without surcharge, where the diameter of the opening is three times the diameter of the plate.

Using the equipment and procedure described, over 200 individual loading tests were made during the summer of 1948, an average of about four tests per 8-hour day.

SPECIAL SUBGRADE BEARING TESTS

In an investigation of this nature it is considered essential to develop as complete knowledge as possible of bearing capacity of the subgrade soil, particularly as influenced by the size of loaded area. Obviously the character of this relation will depend upon the degree of uniformity of the material. A soil mass uniform with respect to composition,

(*Continued on page 200*)

Life Characteristics of Highway Surfaces

BY THE DIVISION OF FINANCIAL AND ADMINISTRATIVE RESEARCH, BUREAU OF PUBLIC ROADS

Reported[1] by FRED B. FARRELL, Chief, Highway Cost Section and HENRY R. PATERICK, Head, Annual Cost Unit

One objective of the road-life study phase of the highway planning surveys is the analysis of service-life characteristics of primary rural highway surfaces. The first comprehensive report on this subject was published in PUBLIC ROADS in 1941. The present article reports further information and compares the original service-life estimates with actual later experience.

The reported analyses are based on 248,783 miles of construction and 129,593 miles of retirement, up to 1946, on rural State or Federal-aid primary systems in 16 States. Estimates of average service lives range from 4½ years for low-type to 27 years for high-type surfaces.

For 12 States represented in both reports, future mileages remaining in service were estimated from the average life analyses in the 1941 study, based on actual experience to 1937. Five-year forecasts compared closely with actual mileages remaining in service in 1942. Greater differences were found between the ten-year forecasts and actual mileages remaining in service in 1946, probably due to the lagging construction program during the war.

On January 1, 1946, the average age of low-type surfaces was 10.6 years; of intermediate types, 8.2 years; and of high types, 13.5 years. Corresponding remaining life expectancies of the mileages in service were 4.5, 7.6, and 12.0 years. Of the mileages in service in 1946, it is estimated that 94 percent of the low-type, 74 percent of the intermediate-type, and 44 percent of the high-type surfaces will require rebuilding by 1956.

Of 92,565 miles of retirements through 1945, 58 percent was resurfaced, 30 percent reconstructed, 10 percent transferred to other public agencies, and 2 percent abandoned. During the war the proportion of resurfacing of high types increased noticeably.

Preliminary studies indicate that, because of salvage value at retirement, the average dollar investment life exceeds the average service life of a surface on a mileage basis. For gravel surfaces constructed in 1920–25 in one State, the average service life of the surface was 5.6 years, while the dollar investment life was 8.6 years.

THE PRESENT STAGE of development of the Nation's highways, roads, and streets is the result of tremendous accomplishments over the past 30 years. Proper management of this vast transportation system requires the collection, analysis, and interpretation of complete factual data relating to the construction, maintenance, operation, and administration of the highway plant. The highway planning surveys, undertaken in the middle 1930's by the State highway departments in cooperation with the Bureau of Public Roads, were conceived for this purpose.

The highway plant itself is the proving ground from which can be obtained the facts relating to past and present performance of various types of highway construction under varying degrees of use. An evaluation of these facts in relation to known conditions is essential in order that plans for orderly future development can be undertaken with confidence. The road-life study phase of the comprehensive highway planning surveys provides the means for obtaining the data upon which such evaluations can be made. Among its objectives is the determination of the annual cost of highways, embracing such items as rates of wearing out, construction costs, maintenance costs, extent of functional obsolescence and structural deterioration, and life of the investment.

Up to the present time, the road-life studies in the individual States have been confined largely to the primary State highway systems or the Federal-aid primary systems. They are gradually being extended to State secondary and Federal-aid secondary systems and to city streets.

The principal progress in this field of research has been in the determination of the life characteristics of highway surfaces. The first comprehensive report on this subject was made in 1941.[2] In that report (hereafter referred to simply as the 1941 report) were included the results of service-life analyses of 210,000 miles of construction up to January 1, 1937, of various surface types in 26 States.

Activity on the road-life study was considerably curtailed during World War II, and the work was not resumed on a broad scale until 1946. Of the 37 States which now have prepared road-life mileage tabulations, there are 16 with a cut-off date of January 1, 1946. The tabulations for the remaining 21 States are not sufficiently current to warrant inclusion in the present analysis.

Data for the following 16 States are included in this report:

Idaho	Nevada
Illinois	New Mexico
Indiana	Oklahoma
Kansas	Texas
Maryland	Utah
Minnesota	West Virginia
Missouri	Wisconsin
Montana	Wyoming

BASIC DATA COMPILED

The basic data compiled for the purpose of this report embrace 248,783 miles of construction of various surface types on the rural portions of the primary State or Federal-aid systems of these 16 States. In general, all mileage in incorporated places of greater than 1,000 persons has been excluded. Construction of widening has also been omitted in those cases where the widening was done as a separate operation. Where the widening was done in conjunction with the resurfacing or reconstruction of the previous surface, the mileage of the new construction, which includes widening, is represented in the tabulations. The basic summaries cover the period from January 1, 1900, to January 1, 1946, and include:

1. Mileage constructed each year for each surface type (for 16 States).

2. Mileage of each year's construction of each surface type remaining in service on January 1 each year after construction (for 16 States).

[1] Presented at the 28th annual meeting of the Highway Research Board, December 1948.

[2] *Life characteristics of surfaces constructed on primary rural highways*, by Robley Winfrey and Fred B. Farrell; PUBLIC ROADS, vol. 22, No. 1, March 1941.

3. Replacement surface types for mileage of each surface type retired each year (for 13 States).

4. Method of retirement (resurfaced, reconstructed, abandoned, or transferred) for mileage of each surface type retired each year (for 13 States).

Data for Idaho, Wisconsin, and Wyoming were not available for the summaries prepared in connection with items 3 and 4.

In addition, there are certain data from three of the States—Missouri, West Virginia, and Wisconsin—relating to dollars invested each year and remaining in service each subsequent year. Some results of these analyses are also included in this report.

There are eight major surface types for which individual service-life analyses were made:

1. Soil surfaced.
2. Gravel or stone.
3. Bituminous surface treated.
4. Mixed bituminous.
5. Bituminous penetration.
6. Bituminous concrete.[3]
7. Portland-cement concrete.
8. Brick or block.

Definitions of these surface types, used in all phases of the highway planning surveys in determining the general type classifications constructed in the individual States, will be found in the appendix, page 196. Definitions of the four methods of retirement will also be found in the appendix.

AVERAGE LIFE DEFINED

The average service life of a road surface is the average period of time after construction that the surface remains in service prior to being replaced, resurfaced, reconstructed, or otherwise taken out of service for any reason or by any method. Stated in another manner, it is that period of time after construction during which the only operations performed on the road surface are those of maintenance as practiced by the various States. It is a recognized fact, however, that a significant amount of construction work is done by maintenance forces and paid for out of maintenance funds in many States. In recording the original data summarized in this report, an attempt was made in each State to segregate construction from maintenance in a uniform manner regardless of the particular accounting practices in effect in a given State. The classifications of construction and maintenance operations generally followed in the road-life study are those included in the tentative draft of the report to the 1938 meeting of the Subcommittee on Uniform Accounting of the American Association of State Highway Officials.[4]

[3] Including sheet asphalt and rock asphalt.

[4] Copies of this tentative draft were transmitted to all State highway departments under date of June 2, 1938, by the Subcommittee on Uniform Accounting, American Association of State Highway Officials.

ANALYSIS PROCEDURES

The survivor curve analysis procedures employed in this report are the same as those which are discussed at some length in the 1941 report. Reference should be made to this earlier report for an explanation of the mechanics of computing average service life. There is one difference in the manner of analyzing the data compiled by the States: In the 1941 report, a single analysis was made of the combined data for all States; in the present report, individual analyses were made for each State and the results combined by weighting. The two procedures will yield the same result, however.

The average life data included in this report represent estimates based upon actual experience. Over the years there have been changes in construction methods and design standards. There have been periods of accelerated activity and periods when little or no construction work was accomplished. Some roads have been kept in service too long while others have been rebuilt before the end of their useful life. Maintenance has frequently been inadequate. There have been many instances of overdesign and underdesign.

Throughout the past 30 years, nevertheless, there has been sustained improvement in the standards of highway design, construction, maintenance, and administration. Each of these has its influence upon service life, but their individual effects cannot be evaluated with certainty. As a result of improvements which are continually being made in design standards, for example, certain factors contributing to early obsolescence or structural failure are gradually being reduced to a minimum, or even eliminated.

It must be recognized, also, that large backlogs of needed replacements of highway facilities have been accumulating for many years. The extent of these backlogs is forcibly brought to attention in the long-range highway needs studies which are under way or which have been recently completed in several States. If the accumulated deficiencies in the highway plant are to be overcome at the rates recommended in these long-range studies, it is likely that the probable remaining service lives may, in some instances, prove to be somewhat less than indicated by the data presented in this report.

While it is true that substantial advances are being made in obtaining greater value from the investment in highways, it should not be concluded that the highway plant can or should be built to last indefinitely. There has been in the past and will continue to be in the future a need for all types of highway facilities. The complexities of modern transportation will also result in shifts in the importance of various roads and streets. Traffic patterns are subject to continual change, and the highway plant must be sufficiently flexible to absorb these changes.

In actual practice only a small percentage of road sections have a life exactly equal to the average. Thus, when there is need for an estimate of service life for a particular road section it is necessary to consider such factors as age, structural condition, design features, location, and traffic usage which relate to that section. Only by the exercise of expert engineering judgment in the evaluation of these factors is it possible to arrive at an estimate of the remaining service life for a particular road section.

MILEAGE IN SERVICE

In table 1 is listed for each surface type, by 5-year construction periods, the mileage constructed during each period and the mileage remaining in service on January 1, 1946. Approximately 38 percent of the surfaced mileage on the primary rural State highway systems is represented by these data, the proportion of each surface type being as follows (*see next page.*):

Table 1.—*Mileages constructed and mileages remaining in service on Jan. 1, 1946, for each surface type* [1]

Construction year period	Soil surfaced		Gravel or stone		Bituminous surface treated		Mixed bituminous		Bituminous penetration		Bituminous concrete		Portland-cement concrete		Brick or block	
	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service
1905 and prior	-------	-------	36.0	0.0	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	0.3	0.0
1906–10	-------	-------	198.7	0.0	-------	-------	-------	-------	4.4	0.0	-------	-------	1.0	0.0	3.7	0.0
1911–15	34.9	0.0	4,330.2	45.8	18.3	9.4	3.7	2.4	29.2	4.2	24.8	7.1	398.5	137.8	65.2	16.4
1916–20	160.0	0.0	6,307.1	146.3	202.1	60.9	182.1	126.4	231.0	42.2	205.8	57.0	1,457.9	654.4	153.6	30.0
1921–25	436.3	7.6	21,521.4	1,194.9	618.7	120.0	1,043.1	640.8	1,290.2	371.6	821.6	297.3	8,854.9	6,651.4	331.2	139.8
1926–30	146.9	18.5	23,176.3	2,177.9	4,104.7	984.6	5,801.0	1,966.1	1,850.9	824.9	1,259.6	559.2	12,486.8	11,637.1	135.1	98.7
1931–35	667.9	48.2	18,998.6	3,376.5	9,300.6	2,782.1	22,470.4	10,970.9	2,886.4	2,041.5	1,521.4	1,014.7	10,581.1	10,331.8	161.8	97.1
1936–40	1,364.7	125.5	10,911.2	3,388.6	12,155.7	8,167.9	22,997.9	15,062.9	1,452.6	1,206.8	2,928.2	2,365.3	4,459.6	4,408.8	23.4	20.4
1941–45	588.5	165.6	4,913.0	3,348.8	8,487.4	7,819.9	7,561.4	7,082.6	1,032.4	1,004.6	3,742.8	3,686.2	1,634.4	1,628.1	14.5	14.5
Total	3,399.2	365.4	90,392.5	13,678.8	34,887.5	19,944.8	60,059.6	35,852.1	8,777.1	5,495.8	10,504.2	7,986.8	39,874.2	35,449.4	888.8	416.9

[1] Compiled from data submitted by 16 States for rural State or Federal-aid primary systems.

	Percent
Soil surfaced	13
Gravel or stone	27
Bituminous surface treated	32
Mixed bituminous	54
Bituminous penetration	23
Bituminous concrete	32
Portland-cement concrete	44
Brick or block	31
Average, all types	38

There are some mileages, particularly of the lower types, for which the date of retirement is known but for which the date of initial construction was not available. This results primarily from the difficulty in locating records of early construction. The partial data in these cases are not included in the analysis.

The probable average service lives for each surface type are shown in table 2, for 5-year construction periods. Estimates of average lives are given in this table only when retirements were sufficient to warrant making the estimates. Because of the smaller mileages involved, the retirement trends for earlier construction are frequently more erratic than the trends for the larger mileages of more recent construction. The average life estimates for this earlier construction are more reliable, however, because of the greater retirement experience.

In addition to the major types listed in table 2, there are certain subtypes for which the average service lives have been computed. Probably the most interesting of these subtypes is old portland-cement concrete which has been resurfaced with mixed bituminous or bituminous concrete surfaces. Of the 2,500 miles of this subtype included with the major type in this report, 77 percent involves old portland-cement concrete resurfaced with bituminous concrete and 23 percent resurfaced with mixed bituminous surfaces. For the major mileage of this subtype, there have been insufficient retirements upon which to base an estimate of average life. During the period 1926–35, however, there were about 230 miles of old concrete resurfaced, of which less than 10 percent was still in service on January 1, 1946. On the basis of this retirement experience, the additional service life obtained from old portland-cement concrete roads after they are resurfaced is 13.7 years. The limited retirement experience for later construction of this subtype indicates that the average service life will remain in the neighborhood of 14 years.

Table 2.—*Weighted probable average service lives for various construction-year periods for each surface type* [1]

Construction-year period	Soil surfaced	Gravel or stone	Bituminous surface treated	Mixed bituminous	Bituminous penetration	Bituminous concrete	Portland-cement concrete	Brick or block
	Years	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*
1905 and prior	-------	19.9	-------	-------	-------	-------	-------	40.0
1906–10	-------	12.0	-------	-------	15.6	-------	27.7	24.7
1911–15	6.3	14.7	23.9	24.4	14.8	24.9	23.9	24.7
1916–20	12.1	10.5	17.9	25.4	15.3	18.3	24.5	21.6
1921–25	8.7	9.1	14.4	21.9	16.2	18.1	26.1	20.2
1926–30	7.2	6.6	9.1	12.3	16.0	15.6	28.2	21.1
1931–35	4.5	5.9	7.4	11.6	15.9	14.7	-------	16.4
1936–40	2.9	5.6	10.3	11.6	14.5	13.5	-------	-------
1941–45	2.1	5.1	-------	-------	-------	-------	-------	-------

[1] Based on analyses of data submitted by 16 States for rural State or Federal-aid primary systems. Average lives shown in this table are to the nearest 0.1 year, but they should not be presumed accurate to this extent. The averages would be materially affected by excluding certain States or by including additional States.

INFLUENCE OF WAR ON RETIREMENTS

The war period has had an influence upon the retirement trends of various surface types. Many surfaces were kept in service beyond their normal life even though subjected, in many cases, to greater wear than at any previous period. In order to reflect the additional life thus obtained in the average life estimates included in this report, it was necessary to make a minor adjustment to the analysis procedures outlined in the 1941 report. This was accomplished by assuming that the backlog of deferred work during the war would be overcome and the normal trend resumed within a maximum of 10 years after the war. The rate of recovery estimated in each instance depended upon the percentage surviving at the beginning of the war, the age of these survivors, the previous rate of retirement, and the retirements, if any, which were made during the war period.

COMPARISON OF TWO STUDIES

There are certain differences between the average lives presented in the 1941 report and those listed in the present report. Table 3 shows a comparison of the average lives for various types for the most recent 5-year periods for which data were listed in the 1941 report.

No particular significance should be attributed to the differences in average lives shown in table 3. Variations in average lives of this magnitude are not uncommon when the analyses are based upon different groupings of States. Further, unusual construction practices in one State may have considerable effect upon the average for a group of States. In Wisconsin, for example, the practice of frequent resurfacing of bituminous surface-treated and mixed-bituminous roads has been

Table 3.—*Comparison of average lives presented in the 1941 report with those listed in the present report*

Surface type	Comparison period [1]	1941 report (26 States)		Present report (16 States)	
		Miles constructed	Average life	Miles constructed	Average life
		Miles	*Years*	*Miles*	*Years*
Soil surfaced	1931–35	[2] 2,542	[2] 5.4	668	4.5
Gravel or stone	1931–35	22,793	6.0	18,999	5.9
Bituminous surface treated	1931–35	[2] 10,286	[2] 11.4	9,301	[3] 7.4
Mixed bituminous	1926–30	5,610	14.3	5,801	[4] 12.3
Bituminous penetration	1926–30	3,725	17.0	1,851	16.0
Bituminous concrete	1921–25	2,362	17.9	822	18.1
Portland-cement concrete	1921–25	6,737	24.4	8,855	26.1
Brick or block	1921–25	980	18.2	331	20.2

[1] The most recent period for which data were presented in table 18 in the 1941 report.
[2] 1931–34.
[3] If Wisconsin data are excluded, average life is 10.2 years.
[4] If Wisconsin data are excluded, average life is 13.0 years.

Table 4.—*Comparison of forecasts of the amounts of surfacing remaining in service with later actual experience* [1]

Surface type	Original mileage constructed [2]	Percentage of original construction remaining in service—			
		As of Jan. 1, 1942		As of Jan. 1, 1946	
		Forecast	Actual experience	Forecast	Actual experience
	Miles	*Percent*	*Percent*	*Percent*	*Percent*
Soil surfaced	781	11	22	2	8
Gravel or stone	40,416	12	14	4	11
Bituminous surface treated	7,420	51	52	34	40
Mixed bituminous	7,449	56	55	37	49
Bituminous penetration	3,533	53	55	37	37
Bituminous concrete	989	45	46	27	38
Portland-cement concrete	3,471	63	69	48	60
Brick or block	408	58	56	42	34
Total	64,467	27	29	16	23

[1] Comparison of data for 12 States included in both the 1941 report and the present report. Forecasts were based on analyses of previous tabulations which showed actual experience to Jan. 1, 1937.
[2] Includes mileage constructed through the latest year for which average life data were presented in the 1941 report.

Table 5.—*Mileages retired for which methods of retirement were determined* [1]

Surface type	Miles retired during each year group						
	1927 and prior	1928–30	1931–33	1934–36	1937–39	1940–42	1943–45
Soil surfaced	88.9	77.2	256.8	308.9	697.2	907.6	431.4
Gravel or stone	4,335.9	8,310.7	14,459.3	12,227.0	9,552.3	6,950.9	1,979.8
Bituminous surface treated	37.0	136.3	802.6	1,488.4	2,435.8	2,240.5	2,705.5
Mixed bituminous	31.2	153.0	832.3	1,448.2	4,603.0	2,759.9	2,455.4
Bituminous penetration	137.5	145.7	271.5	489.5	433.1	689.8	1,023.4
Bituminous concrete	51.1	39.7	170.5	298.8	435.0	615.3	852.1
Portland-cement concrete	109.7	151.6	369.7	347.4	592.4	690.8	1,464.9
Brick or block	18.6	27.0	56.2	35.7	84.1	107.8	143.1
Total	4,809.9	9,041.2	17,218.9	16,643.9	18,832.9	14,962.6	11,055.6

[1] Compiled from data submitted by 13 States for rural State or Federal-aid primary systems.

Table 6.—*Retired mileages for each surface type, and percentage distribution according to method of retirement (total for 1945 and prior)* [1]

Surface type retired	Total retired	Methods of retirement				
		Resurfaced	Reconstructed	Abandoned	Transferred	Total
	Miles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Soil surfaced	2,768.0	68.6	24.5	1.8	5.1	100.0
Gravel or stone	57,815.9	58.8	29.7	2.3	9.2	100.0
Bituminous surface treated	9,846.1	58.9	27.1	3.4	10.6	100.0
Mixed bituminous	12,283.0	51.3	37.3	2.0	9.4	100.0
Bituminous penetration	3,190.5	56.3	24.7	3.7	15.3	100.0
Bituminous concrete	2,462.5	65.9	19.8	2.4	11.9	100.0
Portland-cement concrete	3,726.5	54.8	26.6	3.3	15.3	100.0
Brick or block	472.5	53.3	30.6	2.7	13.4	100.0
Total	92,565.0	58.0	29.8	2.4	9.8	100.0

[1] Computed from data submitted by 13 States for rural State or Federal-aid primary systems.

followed. If the experience for Wisconsin is excluded from the analyses for the 16 States included in this report, the average lives for these types show a closer agreement with the 1941 report.[5]

There are 12 States [6] included in the present report which were included in the 1941 report. For these 12 States it is now possible to compare forecasts of mileages remaining in service, as obtained from analyses made in 1941, with actual experience. In the 1941 report, estimated average lives for the various surface types were presented up to and including the most recent year for which the retirement experience was sufficient to enable a reasonable estimate of the average life to be made. Based upon these previous estimates of average lives, projections were made, for each of the 12 States, of the probable mileages of each surface type that would still be in service on January 1, 1942, and on January 1, 1946. The extent to which these forecasts varied from the actual experience is shown in table 4.

For the most part, the forecasted amounts and the actual amounts remaining in service on January 1, 1942, show reasonable agreement. The greatest difference is in the soil-surfaced type but, because of its short life and limited mileage, its retirement trend is usually erratic. For the other types, the forecasts and the actual amounts remaining in service are in close agreement except for portland-cement concrete where the retirements were somewhat overestimated.

Among the probable reasons for the differences are a lagging highway program and the possible tendency to underestimate the average life upon which the forecast was based. [7] There are doubtless other reasons involved, but their specific influence on the previous analyses are most difficult to determine and hence have not been evaluated.

Greater differences are apparent in the comparison of forecasts and actual mileages remaining in service on January 1, 1946. In this case, the differences can be attributed to a large extent to the lagging highway program during the war period.

The evidence in table 4 would indicate that the analysis procedures employed in the 1941 report yielded generally satisfactory results when appraised in the light of actual experience. With more actual experience in construction and retirements to draw upon, with each successive year, it can be expected that the analysis techniques will yield results progressively more reliable.

[5] The data for Wisconsin were not included in the 1941 report.

[6] Idaho, Indiana, Kansas, Maryland, Missouri, Montana, New Mexico, Oklahoma, Texas, Utah, West Virginia, and Wyoming.

[7] In Bulletin 125 of the Iowa Engineering Experiment Station, *Statistical Analyses of Industrial Property Retirements*, by Robley Winfrey, pp. 86–92, the probable error in determining average life by comparison with the 18 type survivor curves is discussed. In this connection tests were made upon the reliability of the average life estimates, which indicate two significant results: ". . . . First, the average of the under- and over-estimates of probable average life balances close to a zero error; second, the average negative error and the average positive error are about 10 percent at the 70 percent-surviving level and about 15 percent at the 80 percent-surviving level. For certain short curves, however, individual estimates of probable average lives may be greatly in error when compared with the average life determined from the completed curve. Such large errors are caused mainly by two factors: First, often the few data in the upper 10 to 30 percent of the survivor curves are insufficient to definitely fix the curve through this area especially when these few points do not follow a smooth path; second, the succeeding data frequently depart from the trend established by the first few data."

Table 7.—*Percentage retired by various methods for all surface types combined, for various periods* [1]

Method of retirement	Period							
	1927 and prior	1928–30	1931–33	1934–36	1937–39	1940–42	1943–45	Total, 1945 and prior
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Resurfaced	65.2	57.8	51.3	60.4	60.4	52.4	65.7	58.0
Reconstructed	24.3	30.7	34.5	26.0	27.5	35.2	26.0	29.8
Abandoned	1.1	1.1	2.9	2.8	3.1	2.7	1.1	2.4
Transferred	9.4	10.4	11.3	10.8	9.0	9.7	7.2	9.8
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

[1] Computed from data submitted by 13 States for rural State or Federal-aid primary systems.

Table 8.—*Percentage of total retirements by resurfacing for various types for various periods* [1]

Surface type retired	Period							
	1927 and prior	1928–30	1931–33	1934–36	1937–39	1940–42	1943–45	Total, 1945 and prior
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Soil surfaced	83.5	38.3	51.3	75.7	82.3	68.3	54.6	68.6
Gravel or stone	65.8	58.8	51.9	66.7	59.8	56.3	49.6	58.8
Bituminous surface treated	63.2	49.2	52.2	51.7	64.8	53.8	64.4	58.9
Mixed bituminous	51.3	52.4	41.5	33.4	59.8	39.9	62.0	51.3
Bituminous penetration	68.8	47.8	30.7	32.7	46.7	49.8	82.2	56.3
Bituminous concrete	4.5	32.2	70.9	38.5	67.2	56.8	85.8	65.9
Portland-cement concrete	59.5	50.5	54.9	40.1	38.5	36.4	73.6	54.8
Brick or block	26.9	14.5	28.5	18.8	37.3	55.1	90.4	53.3
Total	65.2	57.8	51.3	60.4	60.4	52.4	65.7	58.0

[1] Computed from data submitted by 13 States for rural State or Federal-aid primary systems.

CLASSIFICATION BY RETIREMENT METHOD

In 13 of the 16 States included in the average life analyses, the retired mileages for various years were classified in accordance with the method by which the retirement was made. These retired mileages are listed in table 5.

The methods of retirement into which these mileages were classified are (1) resurfaced, (2) reconstructed, (3) abandoned, and (4) transferred. Definitions of these four general methods of retirement will be found in the appendix, page 196.

Reversion in type, in which a surface reverts to a lower type through lack of adequate maintenance, also represents a distinct method of retirement. Retirements of this nature were so few that they did not warrant consideration in this analysis.

These retirement classifications are general in character and should be so interpreted. Resurfacing is an especially significant method of retirement since it affords an approximate measure of the relative extent to which the various types of surfacing construction are salvaged when they are retired.

In table 6 are shown the total mileages retired and the percentage distribution by methods of retirement. For each surface type, the method of retirement for more than half of all retirements has been by resurfacing. The over-all relationships shown in table 6 have been fairly consistent throughout the years, as is evident in table 7.

RESURFACING INCREASED IN 1943–45

These higher percentages of resurfacing during the period 1943–45 are due in large measure to the curtailed highway program during the war years. Undoubtedly, many miles that ordinarily would have been reconstructed were given a resurfacing treatment to keep them in operation temporarily. Other mileages which would normally have been replaced because of poor location were also kept in service until they could be rebuilt at some future date. Both of these factors unquestionably contributed to reducing the amount of reconstruction and thereby increased the relative amount of resurfacing during this period.

Table 8 shows for each surface type the amounts of resurfacing for various periods of years. It is most interesting to note that the relative amount of resurfacing for mixed bituminous and higher type surfaces was at its greatest value during the period 1943–45. For portland-cement concrete the percentage during this period was double that of the preceding 3 years.

Table 9.—*Mileage in service on Jan. 1, 1946, and estimated percentages which will remain in various future years* [1]

Surface type	In service on Jan. 1, 1946	Remaining in service on— Jan. 1, 1951	Jan. 1, 1956	Jan. 1, 1961	Jan. 1, 1966
	Miles	*Percent*	*Percent*	*Percent*	*Percent*
Low	14,044.2	31.2	6.5	1.1	0.1
Intermediate	55,796.9	58.9	26.0	8.3	2.0
High	49,348.9	80.1	56.0	32.8	15.1
Total	119,190.0	64.4	36.1	17.6	7.2

[1] Based on analyses of data submitted by 16 States for rural State or Federal-aid primary systems.

ANALYSIS BY MAJOR TYPE GROUPS

Figures 1 and 2 show, in graphical form, some of the results of the analyses presented in the preceding pages. For purposes of simplifying these charts, the eight surface types have been combined into three major groups—low, intermediate, and high—as follows:

Low type: Includes soil-surfaced and gravel or stone roads.

Intermediate type: Includes bituminous surface-treated and mixed bituminous roads.

High type: Includes bituminous penetration, bituminous concrete, portland-cement concrete, and brick or block roads.

These combinations are arbitrary and may be criticized in certain respects. Some of the mixed bituminous roads, for example, qualify as high type since they are placed on old rigid bases or on heavy flexible bases of considerable load-bearing capacity. On the other hand, some mileages of the high types are in the intermediate-type category. However, the total mileage of questionable classification is small and considerable effort would be required to insure that each mile was properly classified. Accordingly, no adjustments in this respect were made since the increase in accuracy obtained would not be sufficient to justify such refinement.

Estimates of service lives based upon actual retirement experience were shown in table 2. In order to prepare figures 1 and 2 certain assumptions of average lives are necessary for the recent years. In general, the average lives for the more recent years were assumed to remain about the same or increase slightly (less than 10 percent) in relation to the estimates for the most recent years shown in table 2. These assumptions will no doubt vary somewhat from the actual future experience, but since the bulk of the retirements within the next few years will come from the older construction, any minor differences from the assumed average lives for the more recent construction will not have any major effect upon the over-all trends which are shown.

TRENDS IN SERVICE LIFE

Figure 1 shows, for each 5-year construction period, the trend of mileages of low, intermediate, and high types in service up to January 1, 1946, for the 16 States included in this report, and the rates at which these mileages will go out of service in the future based upon their probable rates of retirement. Table 9 shows the total mileages in service on January 1, 1946, and the probable amounts which will still remain in service for 5, 10, 15, and 20 years in the future.

Table 2 showed the probable average lives for various construction-year periods. These average lives are the expectancies at the time of construction or at age zero. As the road system develops and becomes older, the average age of the surfaces increases and the remaining life expectancy becomes less. Also, as the system becomes older, mileages of earlier retirements are taken out of service, thus leaving in service those mileages whose lives will exceed the average life of the total original construction.

LIFE EXPECTANCY

The probable life of the mileage in service is equivalent to the age plus the expectancy. Under certain conditions it is possible for the average age of mileage in service to exceed the average lives shown in table 2. This is true for low-type surfaced mileage, from which the miles in service have gradually been diminishing since 1931 (see fig. 1). When these low types have been retired, the replacement types have been intermediate or high types. Thus, there has not been sufficient construction of new low types to keep the average age of all low types in service from increasing year to year. As a result, the low types now in service are quite old and have a short remaining life expectancy.

In table 10 are shown the average age, remaining life expectancy, and total probable life of the mileages of low, intermediate, and high types in service, at 5-year intervals from January 1, 1921, to January 1, 1946. This information is also presented graphically in figure 2, and shows strikingly the trends in the

Table 10.—*Average age, life expectancy, and probable life of mileages in service at 5-year intervals* [1]

Date	Low types: Age	Low types: Expectancy	Low types: Probable life	Intermediate types: Age	Intermediate types: Expectancy	Intermediate types: Probable life	High types: Age	High types: Expectancy	High types: Probable life
	Years	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*
Jan. 1, 1921	4.2	8.6	12.8	1.9	19.9	21.8	2.6	20.2	22.8
Jan. 1, 1926	4.0	7.0	11.0	3.5	16.5	20.0	3.2	21.1	24.3
Jan. 1, 1931	5.3	5.3	10.6	2.7	11.1	13.8	4.9	20.4	25.3
Jan. 1, 1936	6.1	5.8	11.9	3.7	10.1	13.8	7.3	18.0	25.3
Jan. 1, 1941	7.9	6.7	14.6	5.2	10.1	15.3	10.4	14.9	25.3
Jan. 1, 1946	10.6	4.5	15.1	8.2	7.6	15.8	13.5	12.0	25.5

[1] Based on analyses of data submitted by 16 States for rural State or Federal-aid primary systems.

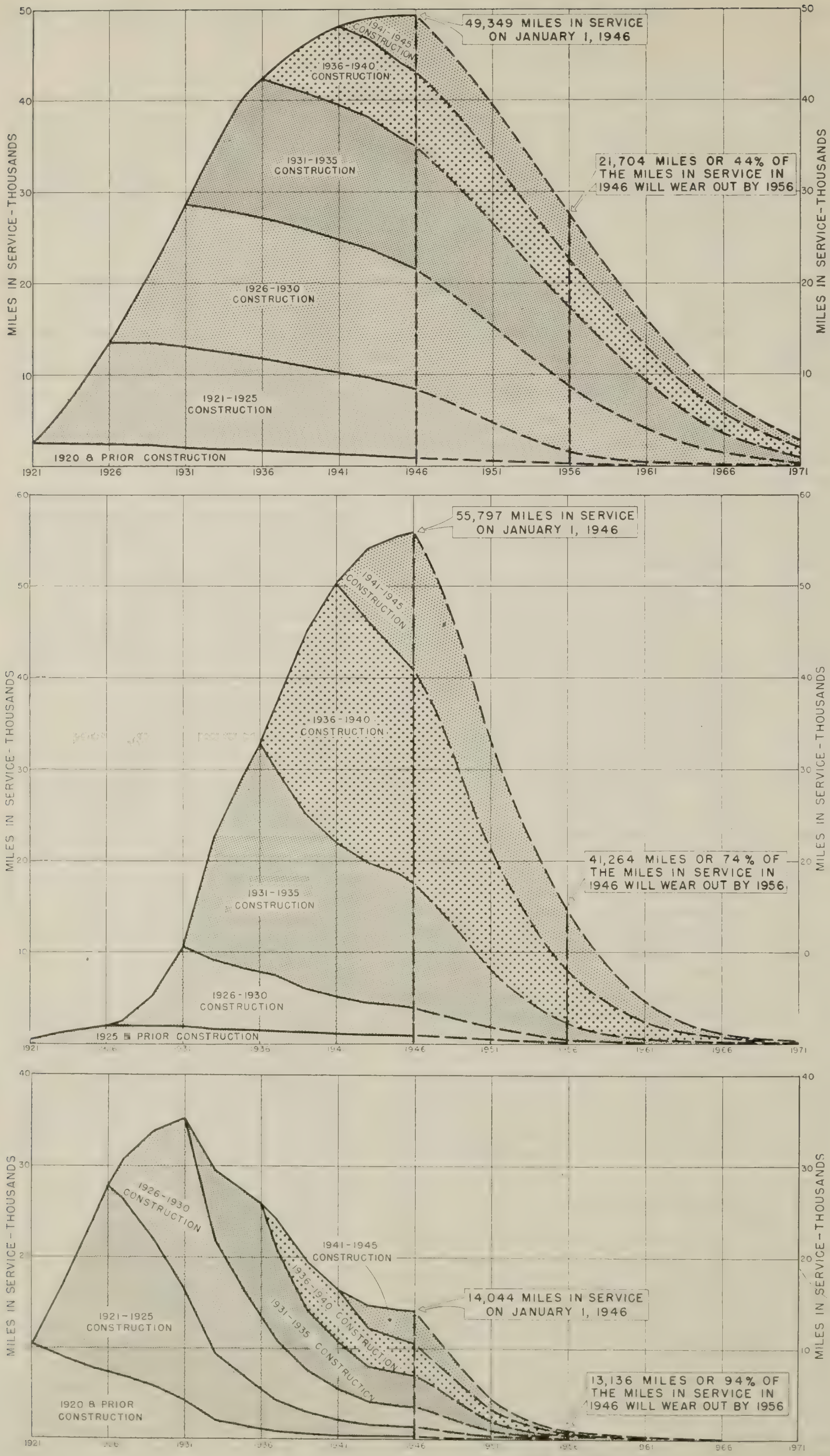


Figure 1.—Of the mileage of primary rural highways in service in 1946:

44 percent of the high-type surfaces will be worn out by 1956;

74 percent of the intermediate-type surfaces will be worn out by 1956;

94 percent of the low-type surfaces will be worn out by 1956.

increasing age and decreasing expectancy of the mileages in service.

LIFE OF INVESTMENT

A phase of the road-life work to which study is currently being given is the life of the dollar investment expended for the construction of highway facilities. This work is now under way in several States and is being conducted in such fashion that a continuous record is maintained of each year's construction investment and the amounts by which this investment is reduced at the time of resurfacing, reconstruction, or other method of retirement. Investment tabulations have now been prepared for Missouri, West Virginia, and Wisconsin.

Because of salvage at the time of retirement, the life of investment will exceed the service life of the surfaced mileage for all surface types. Indications are that the amount will vary anywhere from a few percent for some types having a low salvage at retirement to as much as several hundred percent in those instances involving stage construction or where there is extremely high salvage. During the period 1920–25, for example, approximately $9,500,000 was expended for the construction of 4,800 miles of gravel surfacing on the primary State highway system of Wisconsin. These gravel surfaces had an average life of 5.6 years, but the life of the dollar investment was 8.6 years or more than 50 percent greater than the life of the gravel as a surface. Unfortunately, the retirement experience to date has not been sufficient to warrant presentation in this report of similar estimates of the average life of dollar investment for all years and for each surface type.

It is possible, however, to show a comparison of the mileages constructed and retired with the corresponding investments which entered into their construction and which had been retired by January 1, 1941. From the comparison shown in table 11, it is apparent that the investment in various types is being retired at a slower rate than the mileage.

Since it is a common practice to apply bituminous mats to existing surfaces, the construction investments shown for intermediate-type roads do not reflect the total

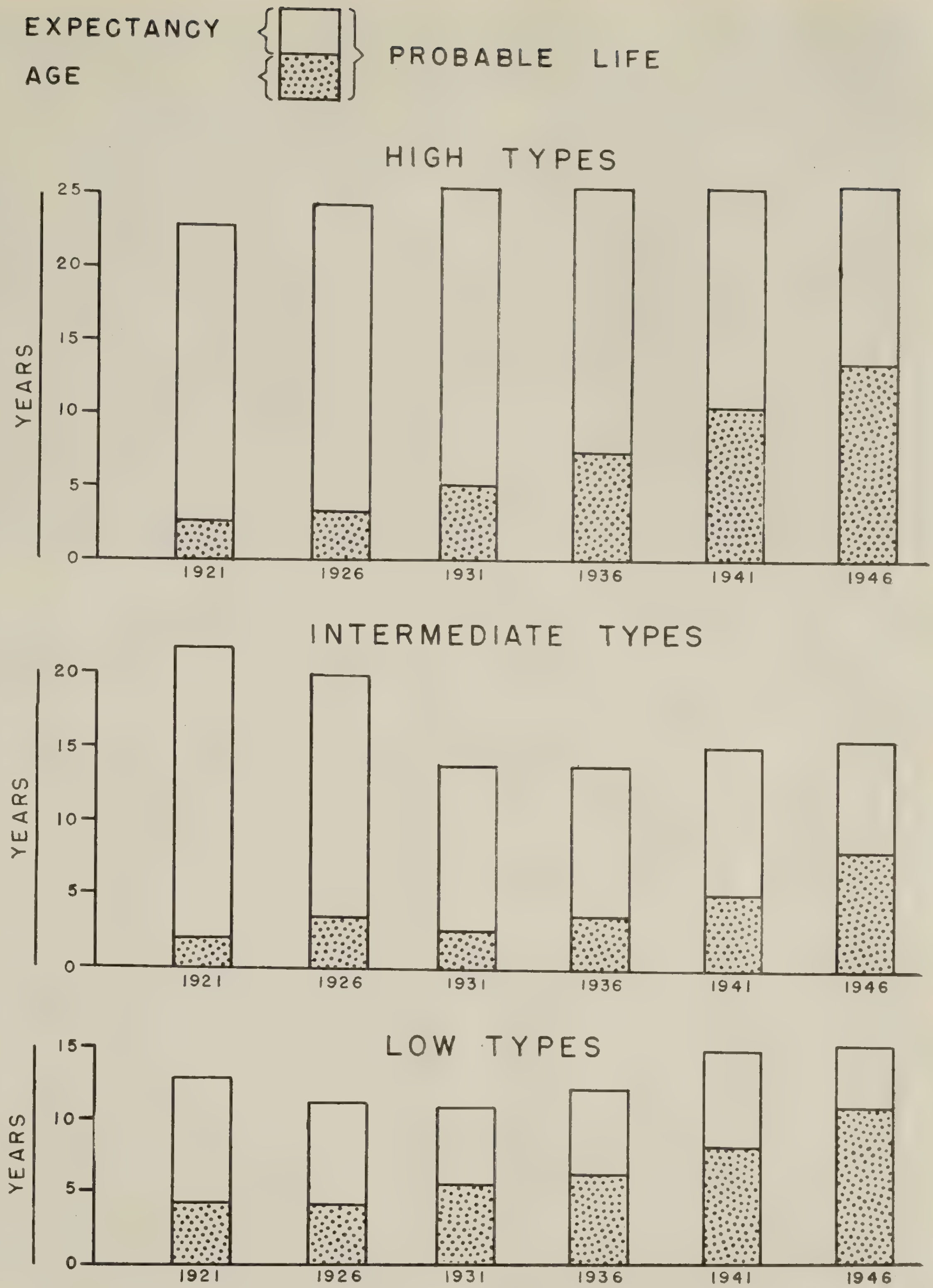


Figure 2.—Trends in average age, expectancy, and probable life, at 5-year intervals: 1921–46.

Table 11.—*Comparison of retirements of mileage with retirements of construction investment* [1]

Surface type as originally constructed	Total construction		Percentage retired up to Jan. 1, 1941	
	Mileage	Investment [2]	Mileage	Investment [3]
	Miles	*Dollars*	*Percent*	*Percent*
Low	21,766	53,300,776	89	57
Intermediate	29,108	[4] 56,304,360	68	26
High	12,489	265,681,512	19	12

[1] Compiled from data submitted by three States for rural State primary highways.
[2] These are actual expenditures for construction. No adjustment is made for the price index.
[3] These represent total losses of prior construction investments, no portion of which is used or usable in the road as of January 1, 1941. No depreciation is included in these percentages.
[4] In the construction of intermediate type, the construction dollars shown fall into two main categories:
(a) If the bituminous mat was placed over an existing low-type surface, the construction dollar investment for the intermediate type includes only the cost of the mat. In these cases there tends to be a high salvage for the low type when it is retired.
(b) If the base and surface mat were built at the same time as a single construction operation, the construction dollar investment for intermediate type includes the cost for both the mat and the base.

costs for a complete road including the base. To obtain the total cost for this type, it would be necessary to take into account the salvage value of gravel or stone and other type roads which may be utilized as a base for the intermediate-type road.

ROAD-LIFE STUDIES ESSENTIAL

The data presented in this report relate only to road surfaces. The road-life studies also embrace research in construction costs, maintenance costs, and salvage values for all elements of the highway, including grading and structures. Knowledge on these subjects will be extended as additional States bring their basic studies up to date and as these studies are continued and extended.

The objective of efficient and economical management of the highway program is to provide highway facilities at such locations and to such standards that they can absorb the inevitable and continuing changes in traffic requirements with the least effect upon the ability of the highway plant to provide maximum service at minimum cost. The data obtained from the road-life studies are among the essential facts needed to reach this objective.

APPENDIX

Surface Type Definitions

Soil-surfaced road.—A road of natural soil, the surface of which has been improved to provide more adequate traffic service by the addition of (1) a course of mixed soil having A–1 or A–2 characteristics, such as sand-clay, soft shale or topsoil, or (2) an admixture such as bituminous material, portland cement, calcium chloride, sodium chloride, or fine granular material (sand or similar material).

Gravel or stone road.—A road the surface of which consists of gravel, broken stone, slag, chert, caliche, iron ore, shale, chat, disintegrated rock or granite, or other similar fragmental material (coarser than sand) with or without sand-clay, bituminous, chemical, or portland-cement stabilizing admixture or light penetrations of oil or chemical to serve as a dust palliative.

Bituminous surface-treated road.—An earth road, a soil-surfaced road, or a gravel or stone road to which has been added by any process a bituminous surface course, with or without a seal coat, the total compacted thickness of which is less than 1 inch. Seal coats include those known as chip seals, drag seals, plant-mix seals, and rock-asphalt seals.

Mixed bituminous road.—A road the surface course of which is 1 inch or more in compacted thickness composed of gravel, stone, sand, or similar material, mixed with bituminous material under partial control as to grading and proportions.

Bituminous penetration road.—A road the surface course of which is 1 inch or more in compacted thickness composed of gravel, stone, sand, or similar material bound with bituminous material introduced by downward or upward penetration.

Bituminous concrete, sheet asphalt, or rock-asphalt road.—A road on which has been constructed a surface course 1 inch or more in compacted thickness consisting of bituminous concrete or sheet asphalt, prepared in accordance with precise specifications controlling gradation, proportions, and consistency of composition, or of rock asphalt. The surface course may consist of combinations of two or more layers, such as a bottom and a top course or a binder and a wearing course.

Portland-cement concrete road.—A road consisting of portland-cement concrete with or without a bituminous wearing surface less than 1 inch in compacted thickness.

Brick[1] *or block road.*—A road consisting of paving brick, stone block, wood block, asphalt block, or other form of block, with or without a bituminous wearing surface less than 1 inch in compacted thickness.

Methods of Retirement

Resurfacing.—Roads which are resurfaced or used as a base for the replacement type are so classified when the old surface is utilized more or less intact (with the exception of necessary scarifying, reshaping, or partial reworking of the surface) in the new construction which retires the old surface. Examples of this method are the retirement of a soil-surfaced road by surface treating, or the retirement of a gravel or stone road by utilizing it as a base or foundation for a mixed bituminous road or a bituminous penetration road. For surfaces which are retired by this method, it is obvious that the new or replacement construction must necessarily be along the same alinement and practically the same grade.

Reconstruction.—When surfaces are retired by reconstruction, there is little or no salvage of the old surface and base into the new type constructed. This classification includes old surfaces and bases that are torn up and not re-used. Usually, for types that are retired by this method, the replacement type is built along the same general alinement (generally within the limits of the existing right-of-way) involving only minor improvements in horizontal curvature. Substantial improvements are usually made with respect to grades, however.

Abandonment.—When the new construction is on new location, the old road is classified as abandoned when it is no longer maintained or kept in service at public expense. The abandoned road may revert to a private road, be barricaded to public travel, or torn up and removed. Sometimes, because of changes in land usage, such as abandonment of factories, and removal or construction of railroad facilities, roads may be abandoned without involving new construction that may be considered as replacing the mileage abandoned.

Transfer.—A retirement by transfer is similar to an abandonment except that the old road is continued in service after being dropped from the State or Federal-aid system by being maintained and resurfaced or reconstructed, when necessary, by the county or other authority responsible for the upkeep of the roads not on the State or Federal-aid system. A transfer is not a retirement in the sense that the road has rendered its total service to the public, but merely that it has rendered its complete service as a primary State or Federal-aid highway. Retirements by transfer are generally the result of functional obsolescence involving alinements and grades which are unsatisfactory for existing traffic conditions. A new road is built on new alinement and improved grades, and the old road remains in service usually because of the necessity of providing for local traffic usage. After the new road is placed in service on the State or Federal-aid highway system, the State will no longer desire to continue responsibility for further upkeep of the old road, and the county or other local authority generally takes over this responsibility. If the road is entirely discontinued from service it is considered an abandonment.

[1] Vitrified paving-brick roads were reported by the States separately from other types of brick or block roads. Because of the small mileages involved, these two types are combined in this report. Approximately 99 percent of the construction of these two types included in the report is vitrified paving brick.

Sheet-asphalt test sections on Connecticut Avenue, Washington, D. C.

Selection of Reference Densities for Bituminous Pavement Specifications

BY THE DIVISION OF PHYSICAL RESEARCH BUREAU OF PUBLIC ROADS

Reported by H. M. REX, Materials Engineer

Few terms in asphalt paving technology are used so indiscriminately and are so liable to misinterpretation as the term "density". As applied to compacted bituminous mixtures, it is used to denote bulk specific gravity, unit weight, and degree of denseness or volume percentage of solids. Similarly, the terms "maximum density" and "theoretical maximum density" are very often used in specifications unaccompanied by sufficient definitive language; in such cases they are open to misunderstanding on the part of job engineers and contractors. Selection of a reasonable relative density value to designate the end-point of compaction during construction is likewise dependent on understanding the explicit meaning of the several expressions.

This paper discusses three different concepts of the expression "maximum density" and shows, by means of illustrative data, the degree to which reasonable relative density values for inclusion in a specification may vary with the respective concepts.

IN writing construction specifications for the various types of bituminous roadway surfaces, particularly those involving the hot plant-prepared mixtures, a relative density value is often used to delineate an end-point of compaction by the roller equipment. As usually written, such specifications require that rolling be continued until the bituminous surface has attained a specified percentage of a variously described maximum density. Sometimes the reference density is designated as that of a specimen, composed of the same mixture as used in the roadway, compacted in the laboratory. More often, however, the reference density is described merely as a "maximum density," or as a "theoretical maximum density."

The terms "maximum density" and "theoretical maximum density" are sometimes used synonymously. To many, however, the terms are considered as having quite different connotations. A "theoretical maximum density" is generally understood to be the calculated specific gravity of a voidless mass composed of bitumen and aggregate in the same relative proportions as used in the designed pavement mixture. By way of differentiation, the term "maximum density" is often used in referring to the observed specific gravity value of a laboratory-compacted specimen of the mixture, or a calculated specific gravity value based on the unit weight of the aggregate only, as determined by vibratory compaction in the laboratory. The numerical value obtained by any of these methods may be considered as being a "maximum density." When the value is derived (1), by calculating the specific gravity of an imaginary mass or (2), by using the unit weight of vibrated aggregate as the basis for computation, such a value could also come within the meaning of a "theoretical maximum density."

Which of the methods is employed in arriving at a reference specific gravity is less important than that the reference specific gravity be adequately identified in the specifications, and that the percentage of this reference specific gravity, or the relative density, to be required in the compacting process during construction be selected from a realistic viewpoint.

In most cases the use of a specific gravity value of laboratory-compacted specimens of the mixture as the reference maximum density will be found to be most satisfactory from a practical viewpoint. This procedure will, of course, entail a certain amount of work in ascertaining the compactive effort required with the use of laboratory equipment to produce the highest density for a given mixture without causing, at the same time, excessive crushing of the aggregate particles.

In laboratories equipped with suitable vibratory compaction equipment,[1] an equally

[1] *A new vibratory machine for determining the compactibility of aggregates*, by J. T. Pauls and J. F. Goode, PUBLIC ROADS, vol. 20, No. 3, May 1939.

Method of cutting cores for laboratory testing.

satisfactory method consists in consolidating to maximum density the uncoated aggregate only, and using the unit weight so obtained, with the specific gravity of the aggregate, in computing the reference maximum density value. The manner in which each of these laboratory procedures is used to develop a reference maximum density value will be fully explained later in this discussion.

RELATIVE DENSITY REQUIREMENT SHOULD BE PRACTICAL

When the calculated specific gravity of a theoretical voidless mass is used as the reference point for specification purposes, the relative density required for satisfactory compaction during construction should be selected with the greatest attention to practical considerations. If this is not done the limit may be so high as to produce some very undesirable effects in the pavement structure or, indeed, as to be impossible of attainment. For example, assume that an aggregate intended for use in bituminous concrete contains, in its densest possible arrangement of particles, 22 percent of voids, and that the optimum bitumen content appears to be 14 percent by volume. Assume also that the minimum relative density of the pavement, as compacted during construction, is stated by the specifications to be 95 percent of the specific gravity of a voidless mass. Manifestly this relative density could only be obtained, if it could be obtained at all, by crushing the aggregate to a distinctly undesirable degree.

Excellent illustrative information relating to this subject of density is provided by some incidental data collected in connection with a laboratory and field study of the alterations in a sheet-asphalt pavement resulting from age. This study was undertaken cooperatively by the District of Columbia Department of Highways and the Bureau of Public Roads, and was initiated in 1935–36. In all, 10 test sections of sheet-asphalt pavement were constructed on Connecticut Avenue in Washington, D. C., one of the city's most heavily traveled streets. The test sections were characterized by a number of mixture design variables. Compaction was not included as a variable, hence the construction procedures followed in all the sections were the same as those followed in usual sheet-asphalt contract work in that city. Cores, 2 inches in diameter, were cut from each of the sections shortly after construction and periodically thereafter over a 10-year period. Eighteen cores were cut from each section at each sampling. These cores were tested in the laboratory in a number of ways, the specific gravities of the top inch of each core being included in the test results. The illustrative values given in this discussion consist of the average of each set of 18 specimens cut from the 10 sections.

The design compositions, by weight, of the mixtures used in the 10 sections are given in the first part of table 1, together with the specific gravities of the asphalts. The apparent specific gravity of the combined mineral aggregate showed little variation within the range of weight proportions of sand and filler used. Many tests established a value of 2.674 for these combined materials.

COMPUTATION OF VOLUMETRIC COMPOSITION

Blends of sand and filler used in the mixtures were tested in the laboratory for vibratory density, and the unit weights of the vibrated aggregate are shown in the fifth column of table 1. Using these unit weights, together with the weight proportions and specific gravities of the asphalts, it is possible to calculate the volumetric composition, including air voids, of the mixtures theoretically compacted to their densest condition. These values are shown in the central part of table 1.

The following example will serve to illustrate the method used in computing these values. The unit weight of the aggregate used in section No. 1 was found, by vibratory test, to be 1.89 grams per cubic centimeter, or 189 grams per 100 cubic centimeters. The volume of the aggregate, since its specific gravity is 2.674, will be $189 \div 2.674 = 70.7$ cubic centimeters. The proportions by weight of aggregate and asphalt in this section were 89 and 11 percent, respectively, hence the total weight of 100 cubic centimeters of such a mixture would be $189 \div 0.89 = 212$ grams. Subtracting 189 from 212 grams, the weight of asphalt is 23 grams. As shown in the column of specific gravities in table 1, the weight of a cubic centimeter of asphalt is 1.030 grams, hence the volume of asphalt occupied in 100 cubic centimeters of this mixture is $23 \div 1.030 = 22.3$ cubic centimeters. Thus, in a volume of 100 cubic centimeters of

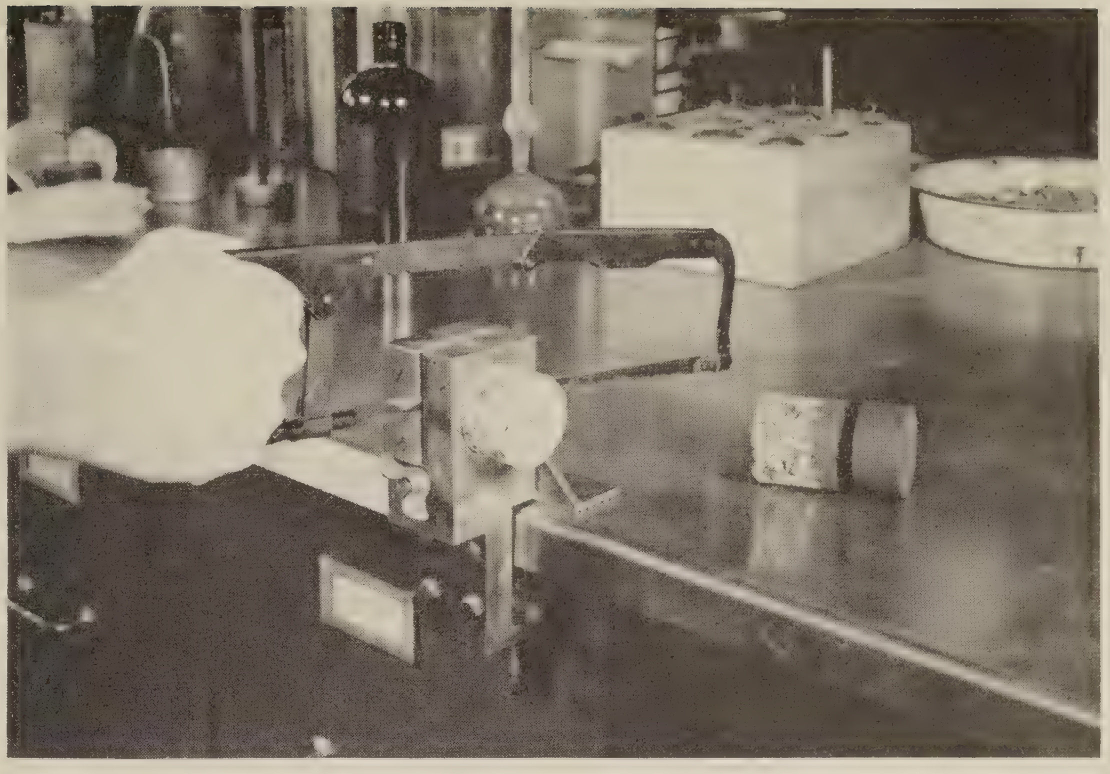
Table 1.—*Composition of mixtures, by weight and by volume*

Mix No.	Specific gravity[1] of asphalt 25/25° C.	Design composition, by weight		Composition by volume, based on unit weight of aggregate compacted by vibration				Composition by volume, based on unit weight of mixtures compacted in laboratory[2]				
		Asphalt	Aggregate	Unit weight of vibrated aggregate	Composition: Asphalt	Composition: Aggregate	Composition: Air voids	Unit weight of compacted mixture	Unit weight of voidless mass[3]	Composition: Asphalt	Composition: Aggregate	Composition: Air voids
		Percent	*Percent*	*Gm. per cm.*[3]	*Percent*	*Percent*	*Percent*	*Gm. per cm.*[3]	*Gm. per cm.*[3]	*Percent*	*Percent*	*Percent*
1	1.030	11.0	89.0	1.89	22.3	70.7	7.0	2.12	2.275	22.6	70.5	6.9
2	1.032	11.0	89.0	1.91	23.2	71.4	5.4	2.15	2.276	22.9	71.5	5.6
3	1.026	11.0	89.0	1.89	22.4	70.7	6.9	2.11	2.273	22.6	70.2	7.2
4	1.032	10.5	89.5	1.86	21.3	69.6	9.1	2.08	2.291	21.2	69.6	9.2
5	1.028	11.0	89.0	1.89	22.4	70.7	6.9	2.12	2.274	22.7	70.6	6.7
6	1.027	11.0	89.0	1.90	22.4	71.0	6.6	2.13	2.273	22.8	70.9	6.3
7	1.030	10.5	89.5	1.89	21.4	70.7	7.9	2.10	2.290	21.4	70.3	8.3
8	1.024	10.5	89.5	1.90	21.5	71.0	7.5	2.09	2.287	21.4	70.0	8.6
9	1.022	11.0	89.0	1.91	23.5	71.4	5.1	2.10	2.270	22.6	69.9	7.5
10	1.022	10.5	89.5	1.87	21.5	69.9	8.6	2.11	2.286	21.7	70.6	7.7

[1] The specific gravity of the aggregate, in all mixes, was 2.674.
[2] Compaction using load of 3,000 pounds per square inch.
[3] Calculated.

this mixture, assuming the aggregate to be compacted to the density indicated in the vibratory test, there are, theoretically, 70.7 cubic centimeters of aggregate solids and 22.3 cubic centimeters of asphalt. There will be, then, $100-(70.7+22.3)=7.0$ cubic centimeters of air voids. Similar computations for the other nine sections indicate minimum air void contents ranging from 5.1 to 9.1 percent. In other words, the maximum relative densities of the sections, according to this type of concept, range from 90.9 to 94.9 percent of corresponding theoretical densities of a voidless mass.

Where equipment to make the vibratory test on the aggregate itself is not available, it is possible to use another method of arriving at a reasonable maximum density reference value. Cylindrical specimens of the mixture may be molded in the laboratory, using the same proportions of aggregate and asphalt as are to be used in actual construction. As a general rule, a static pressure of 3,000 pounds per square inch has been found to be a satisfactory molding load. Experimentation will indicate if deviation from this pressure is required for the particular materials that are to be used.



Sawing top inch of sheet-asphalt cores for density determination.

The application of this method in calculating maximum densities for the mixtures of the sheet-asphalt test sections is illustrated by the results shown in the last part of table 1. In this case cylinders, 2 inches in diameter and 1 inch in height, of each of the 10 design mixtures were molded while hot in the laboratory, using a pressure of 3,000 pounds per square inch. Specific gravity tests on the molded specimens established the unit weights for the 10 mixtures shown in the table.

The other values were computed as follows:

$$U_v=\frac{100}{\frac{W}{G}+\frac{W_1}{G_1}}$$

$$A_v=\frac{UW_1}{G_1}$$

$$C_v=\frac{UW}{G}$$

$$E=100-(A_v+C_v)=100-\frac{100U}{U_v}$$

Where:

U_v = unit weight of a voidless mass (theoretical maximum density), gm. per cm³.

W = percentage of asphalt, by weight.

W_1 = percentage of combined filler and sand, by weight.

G = specific gravity of asphalt.

G_1 = apparent specific gravity of combined filler and sand.

A_v = percentage of aggregate, by volume, in the compacted mixture.

U = unit weight (bulk specific gravity) of the compacted mixture, gm. per cm³.

C_v = percentage of asphalt, by volume, in the compacted mixture.

E = percentage of air voids.

The highest possible densities for the 10 mixtures calculated by this method were found to range from 90.8 to 94.4 percent of voidless mass specific gravities. Although these values are slightly lower than those calculated on the basis of vibrated aggregate densities, the differences are small.

Table 2.—*Densities of pavement samples cored at time of construction and 10 years later compared with maximum specific gravities calculated by different methods*

Mix No.	Maximum specific gravities			Relative densities of actual pavement samples					
	Based on vibratory test results	Based on laboratory compacted specimens	Based on assumption of voidless mass	Vibrated aggregate basis		Compacted mix basis		Voidless mass basis	
				Original	After 10 years	Original	After 10 years	Original	After 10 years
				Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1	2.12	2.12	2.275	97.2	100.0	97.2	100.0	90.5	93.2
2	2.15	2.15	2.276	94.0	99.5	94.0	99.5	88.8	94.0
3	2.12	2.11	2.273	97.6	98.6	98.1	99.0	91.1	91.9
4	2.08	2.08	2.291	97.1	99.5	97.1	99.5	88.2	90.4
5	2.12	2.12	2.274	93.4	100.0	93.4	100.0	87.1	93.2
6	2.13	2.13	2.273	96.2	100.0	96.2	100.0	90.2	93.7
7	2.11	2.10	2.290	96.2	99.0	96.7	99.5	88.6	91.3
8	2.12	2.09	2.287	95.3	99.0	96.6	100.5	88.3	91.8
9	2.15	2.10	2.270	95.8	99.1	98.1	101.4	90.7	93.8
10	2.09	2.11	2.286	95.7	100.5	94.8	99.5	87.5	91.9
Average	------	------	------	95.8	99.5	96.2	99.9	89.1	92.5

RELATION OF UNIT WEIGHTS TO RELATIVE DENSITIES

In table 2 are shown the values for theoretical maximum specific gravities or unit weights as computed on the three different bases, together with the relative densities of the top portions of cores from the pavement sections as sampled just after construction and again after 10 years of service. Only the section composed of mixture No. 2 attained a relative density of as high as 94 percent of a theoretical voidless mass, and that only after 10 years of service. On such a basis, all sections would have failed to pass specifications requiring a minimum relative density of 94 percent at the time of construction. It is likely that attempted enforcement of such requirements would have resulted in crushing of the aggregate particles or laminating and tearing of the pavement surface under the roller.

By contrast, all but one of the sections would have passed such a relative density requirement, if the maximum reference density were based on either compacted mixture specimens or aggregate compacted by vibration. Also, since maximum densities calculated on these two latter bases agree so closely with the observed ultimate densities of samples from the pavement, it would appear that specification requirements written around such maximum densities would be both logical and realistic.

Before stipulating field compaction density requirements, therefore, the specification writer should first of all decide upon a base or reference value with which the density of field samples taken during construction is to be compared. As has been stated above, either of the terms "maximum density" or "maximum theoretical density" may be used to denote the reference value. The value itself may consist of (1) the calculated specific gravity of a theoretical voidless mass, (2) the observed specific gravity of a test specimen of the mixture compacted in the laboratory, or (3) a calculated specific gravity based on the unit weight of uncoated aggregate consolidated by vibration. In any case, the language used in the specifications should be explicit; as, for example, "samples taken from the pavement after compaction shall have a specific gravity not less than -- percent of the maximum density that has been determined for laboratory-compacted specimens of the mixture."

Selection of a minimum relative density value will depend upon the nature of the reference maximum density. In general, relative densities ranging from 95 to 98 percent of the maximum densities may be used if the latter are calculated on the basis of either laboratory-compacted mixtures or unit weights of aggregate vibrated in the laboratory. Considerably lower relative density values may be required if the calculated specific gravity of a theoretical voidless mass of the mixture is used as the reference maximum density. This will be particularly true in the case of surfaces composed of fine-grain mixtures, in some of which, as has been shown, it is possible to find air-void contents as high as 10 percent even after 10 years of service.

(Continued from page 188)

density, and moisture to a considerable depth is certain to exhibit area effects which differ from those found where such uniformity does not exist.

In the investigation being reported a special effort is being made to develop data on this question by testing the 5-foot embankment of uniform A-6 soil with load areas ranging in size from 1 to 7 feet in diameter. In addition to steel plates of 12, 18, 24, and 30 inches in diameter, concrete disks 42, 60, and 84 inches in diameter, as shown in figure 1, are being used as bearing surfaces. In these tests the incremental repetitional procedure is being employed.

In addition to the load-bearing tests there is under way a comprehensive study of temperature conditions in the pavement and subgrade. Sixteen thermocouples have been installed at selected locations and depths so that temperature gradients can be obtained in each pavement test section and in the subgrade beneath, to a maximum depth of 3 feet from the surface of the pavement. These temperatures are recorded at 1-minute intervals by an automatic instrument to an accuracy of $\pm 0.6°$ F. A view of the recorder is shown in figure 8. The locations of the thermocouples in the various pavement sections are shown in figure 2.

SUMMARY

As stated in the introduction, this report is presented primarily for the purpose of describing some innovations in plate-load testing techniques and testing equipment. The need for a rapid method of testing, a method which will permit the conduct of a number of tests in a relatively short period of time, has long been apparent. The method described, which gives, in addition to complete load-deflection relations, information pertaining to the behavior of the components of a flexible pavement structure, may serve partially to fulfill this need.

Highway Needs of the National Defense, a New Publication

Highway Needs of the National Defense, a report compiled by the Bureau of Public Roads in compliance with section 2 of the Federal-aid Highway Act of 1948, which directed the Commissioner of Public Roads to cooperate with the State highway departments in making a detailed study of the status of improvement and principal deficiencies of the National System of Interstate Highways, is being published as House Document 249, Eighty-first Congress, and will soon be available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C. The report is based on detailed data supplied by the State highway departments and embodies suggestions and recommendations of the Secretary of Defense.

The 40,000-mile interstate system is the trunk-line highway system of the United States, connecting most of the large cities of the country and serving the principal industrial centers. Including only 1 percent of the Nation's total road mileage, the system's rural sections serve 20 percent of the total traffic carried by all rural roads. Its urban sections, as thus far designated, serve more than 10 percent of the traffic moving on city streets. These main highways carry the bulk of the Nation's traffic and should be among the first considered for improvement. Large portions of the system are seriously obsolescent and are not suitable for the great number of vehicles attempting to use them.

The estimated cost of proposed improvements to correct existing deficiencies and to adapt the system to the needs of its present traffic is $11,266,000,000, based on 1948 construction costs. Of this sum, $5,293,000,000 would be expended on sections of the system in urban areas, and $5,973,000,000 would be assigned to projects on rural sections. As improvements are undertaken, the report recommends, ample provision should be made for the increased traffic volumes anticipated over a 20-year period. In this period, improvement must be undertaken on every mile of the system, requiring a capital investment of more than $500,000,000 a year. Completion of improvements on the system in a period far shorter than 20 years would result in much greater benefits.

The report advises the Congress to consider authorization of Federal appropriations earmarked for expenditure on the interstate system, apportioned among the States so as to permit substantially equal progress in correcting deficiencies in all States. In view of the interstate interest attaching to the system, the report recommends Federal participation in a ratio greater than the normal 50 percent of the cost. Federal authorizations for the Federal-aid primary, urban, and secondary systems should continue at rates not less than those established in the Federal-aid Highway Act of 1948, the report recommends; and the proposed authorizations for the interstate system should be in addition to these.

The report recommends that future highway legislation should provide for national emergencies in war or peace by authorizing diversion of highway appropriations to other purposes in time of need, such as construction of wartime-needed access roads and the emergency repair of roads damaged by floods and other major disasters. Road-building materials should be stock-piled as a precaution against possible need in the event of war.

U. S. GOVERNMENT PRINTING OFFICE: 1949

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Status of Federal-aid highway program as of June 30, 1949

[Thousand dollars]

State	Unprogramed balances	Active program: Programed only: Total cost	Programed only: Federal funds	Programed only: Miles	Plans approved, construction not started: Total cost	Plans approved: Federal funds	Plans approved: Miles	Construction under way: Total cost	Construction under way: Federal funds	Construction under way: Miles	Total: Total cost	Total: Federal funds	Total: Miles
Alabama	$12, 488	$10, 728	$5, 432	186. 4	$3, 456	$2, 152	156. 9	$11, 395	$6, 163	277. 2	$25, 579	$13, 747	620. 5
Arizona	966	5, 478	3, 813	78. 3	198	140	14. 0	7, 203	4, 903	53. 7	12, 879	8, 856	146. 0
Arkansas	5, 346	9, 850	5, 114	299. 4	8, 227	3, 818	112. 6	9, 269	4, 994	149. 7	27, 346	13, 926	561. 7
California	4, 812	28, 168	9, 698	156. 8	4, 629	2, 111	57. 6	49, 555	24, 211	256. 7	82, 352	36, 020	471. 1
Colorado	3, 966	6, 643	3, 789	214. 7	3, 230	1, 947	76. 0	10, 307	6, 098	156. 9	20, 180	11, 834	447. 6
Connecticut	1, 542	10, 948	5, 130	19. 1	3, 140	1, 520	11. 1	9, 608	4, 966	14. 7	23, 696	11, 616	44. 9
Delaware	2, 714	1, 342	671	11. 8	2, 141	1, 045	21. 8	3, 281	2, 179	24. 2	6, 764	3, 895	57. 8
Florida	3, 968	20, 373	10, 685	504. 4	2, 178	1, 103	69. 9	6, 839	2, 863	114. 4	29, 390	14, 651	688. 7
Georgia	6, 861	12, 765	6, 752	319. 7	12, 477	5, 085	57. 6	34, 796	17, 555	775. 8	60, 038	29, 392	1, 153. 1
Idaho	4, 774	7, 416	4, 627	393. 0	1, 643	984	61. 5	5, 098	3, 214	96. 5	14, 157	8, 825	551. 0
Illinois	5, 659	63, 981	34, 071	707. 5	30, 324	13, 318	222. 9	45, 401	22, 653	499. 5	139, 706	70, 042	1, 429. 9
Indiana	9, 481	23, 760	11, 914	102. 9	3, 543	1, 983	19. 0	21, 894	11, 444	144. 5	49, 197	25, 341	266. 4
Iowa	2, 069	16, 429	7, 611	719. 8	12, 520	5, 677	408. 6	19, 569	9, 481	752. 9	48, 518	22, 769	1, 881. 3
Kansas	3, 379	13, 884	6, 804	1, 551. 5	7, 807	4, 009	490. 0	18, 920	9, 695	918. 2	40, 611	20, 508	2, 959. 7
Kentucky	3, 494	12, 885	6, 523	279. 0	6, 119	3, 068	102. 3	15, 599	7, 860	195. 1	34, 603	17, 451	576. 4
Louisiana	3, 397	24, 593	11, 590	282. 3	12, 563	5, 836	89. 0	17, 223	8, 182	136. 9	54, 379	25, 608	508. 2
Maine	3, 253	6, 572	3, 594	70. 0	864	434	8. 5	7, 522	3, 724	70. 8	14, 958	7, 752	149. 3
Maryland	1, 773	2, 953	1, 395	23. 2	9, 522	3, 267	20. 8	17, 032	9, 282	68. 8	29, 507	13, 944	112. 8
Massachusetts	15, 178	17, 065	9, 352	52. 5	4, 136	2, 153	12. 3	20, 215	10, 603	38. 0	41, 416	22, 108	102. 8
Michigan	2, 991	18, 177	8, 991	506. 6	10, 378	5, 196	283. 7	45, 738	19, 664	297. 0	74, 293	33, 851	1, 087. 3
Minnesota	2, 301	11, 890	6, 370	672. 9	9, 512	5, 048	342. 1	22, 783	11, 474	504. 3	44, 185	22, 892	1, 519. 3
Mississippi	8, 279	2, 740	1, 315	107. 8	3, 914	1, 977	146. 7	16, 615	8, 420	404. 9	23, 269	11, 712	659. 4
Missouri	7, 652	29, 897	14, 995	834. 1	13, 177	6, 006	294. 4	26, 227	14, 071	576. 1	69, 301	35, 072	1, 704. 6
Montana	9, 001	8, 138	4, 998	321. 3	4, 135	2, 618	111. 2	12, 088	7, 270	293. 0	24, 361	14, 886	725. 5
Nebraska	5, 003	17, 543	9, 357	585. 8	5, 099	2, 517	138. 0	10, 116	5, 546	354. 5	32, 758	17, 420	1, 078. 3
Nevada	3, 287	2, 900	2, 372	120. 5	1, 741	1, 432	17. 4	3, 796	3, 126	144. 3	8, 437	6, 930	282. 2
New Hampshire	1, 414	6, 099	3, 019	55. 1	303	147	1. 0	3, 539	2, 062	16. 4	9, 941	5, 228	72. 5
New Jersey	1, 522	8, 305	3, 537	25. 6	5, 118	2, 626	2. 9	32, 307	15, 921	41. 0	45, 730	22, 084	69. 5
New Mexico	2, 072	10, 434	6, 768	364. 7	2, 662	1, 708	47. 9	4, 193	2, 680	68. 3	17, 289	11, 156	480. 9
New York	33, 734	58, 952	32, 163	240. 0	29, 031	13, 723	47. 8	86, 717	42, 921	346. 4	174, 700	88, 807	634. 2
North Carolina	11, 550	6, 703	3, 174	115. 2	3, 193	1, 465	30. 8	24, 027	11, 827	565. 7	33, 923	16, 466	711. 7
North Dakota	3, 563	9, 724	5, 046	1, 326. 7	4, 785	2, 381	304. 2	10, 675	5, 621	629. 8	25, 184	13, 048	2, 260. 7
Ohio	14, 892	42, 181	20, 456	274. 3	6, 524	3, 322	39. 3	54, 163	27, 294	212. 8	102, 868	51, 072	526. 4
Oklahoma	4, 300	20, 481	11, 432	664. 5	10, 524	5, 334	421. 5	13, 643	6, 635	630. 3	44, 648	23, 401	1, 716. 3
Oregon	3, 254	6, 458	3, 541	96. 1	1, 130	608	9. 1	11, 426	5, 317	145. 1	19, 014	9, 466	250. 3
Pennsylvania	11, 070	32, 777	16, 761	102. 3	12, 073	5, 993	32. 9	63, 977	32, 013	145. 1	108, 827	54, 767	280. 3
Rhode Island	2, 509	7, 033	3, 174	20. 3	4, 525	2, 241	7. 1	3, 604	1, 777	10. 5	15, 162	7, 192	37. 9
South Carolina	6, 017	2, 992	1, 547	54. 4	1, 253	663	99. 2	9, 798	5, 065	236. 3	14, 043	7, 275	389. 9
South Dakota	1, 699	10, 066	5, 841	1, 028. 8	4, 986	3, 270	349. 6	10, 434	6, 240	641. 4	25, 486	15, 351	2, 019. 8
Tennessee	3, 625	12, 127	6, 063	447. 9	7, 286	3, 540	187. 3	23, 812	11, 848	258. 6	43, 225	21, 451	893. 8
Texas	9, 028	6, 556	3, 084	284. 6	32, 157	9, 266	726. 5	57, 448	29, 836	1, 081. 6	96, 161	42, 186	2, 092. 7
Utah	1, 570	6, 110	4, 494	226. 7	1, 290	963	54. 9	4, 565	3, 327	110. 0	11, 965	8, 784	391. 6
Vermont	1, 166	2, 649	1, 322	50. 3	1, 397	699	16. 2	4, 153	2, 022	50. 8	8, 199	4, 043	117. 3
Virginia	11, 544	11, 720	5, 889	314. 8	3, 885	1, 891	116. 5	13, 395	6, 287	185. 6	29, 000	14, 067	616. 9
Washington	1, 666	15, 215	6, 555	165. 9	5, 244	2, 131	91. 4	13, 315	6, 395	108. 8	33, 774	15, 081	366. 1
West Virginia	2, 478	10, 279	5, 143	134. 6	3, 434	1, 715	42. 0	10, 585	5, 234	132. 0	24, 298	12, 092	308. 6
Wisconsin	7, 815	23, 327	11, 849	681. 6	5, 288	2, 439	174. 1	26, 749	12, 264	405. 0	55, 364	26, 552	1, 260. 7
Wyoming	408	3, 638	2, 441	99. 4	2, 183	1, 368	142. 9	8, 425	5, 445	300. 1	14, 246	9, 254	542. 4
Hawaii	2, 273	6, 071	3, 043	24. 0	2, 416	1, 035	8. 0	3, 507	1, 847	19. 1	11, 994	5, 925	51. 1
District of Columbia	190	3, 874	2, 149	1. 3	751	375	1. 2	14, 400	6, 931	2. 0	19, 025	9, 455	4. 5
Puerto Rico	3, 082	9, 946	4, 502	45. 8	2, 228	1, 007	12. 0	7, 920	2, 753	37. 8	20, 094	8, 262	95. 6
Total	276, 075	720, 835	369, 956	15, 966. 2	330, 339	154, 354	6, 312. 2	984, 866	499, 203	13, 699. 1	2, 036, 040	1, 023, 513	35, 977. 5