PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 7, NO. 4 ▽ JUNE, 1926

THE ROUGH STONE BLOCK TEST HIGHWAY USED IN THE IMPACT TESTS

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1926

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VOL. 7, NO. 4 JUNE, 1926 H. S. FAIRBANK, Editor

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MOTOR TRUCK IMPACT AS AFFECTED BY TIRES, OTHER TRUCK FACTORS, AND ROAD ROUGHNESS

Report of Cooperative Tests by the Bureau of Public Roads, the Society of Automotive Engineers, and the Rubber Association of America

Reported by **JAMES A. BUCHANAN**, Bureau of Public Roads, and **J. W. REID**, Rubber Association of America

SINCE 1923 there has been in progress at the experimental station of the Bureau of Public Roads at Arlington, Va., a series of motor-truck impact tests designed especially to develop information with respect to the influence of various types of tires, certain elements of vehicular design and equipment, the loading, capacity, and speed of the vehicles, and the roughness of the road surface. After ascertaining the magnitude of the impact under the various conditions created it is the further object of the tests to determine the stresses induced in the road surface. Up to this time, however, efforts have been directed principally toward the attainment of the first objective, and it is to this phase of the investigation that the present report is confined.

The entire investigation is being conducted cooperatively by the Bureau of Public Roads, the Society of Automotive Engineers, and the Rubber Association of America, under the direction of a joint committee representing the three organizations.

Owing to the necessity of developing an accelerometer adapted to the conditions of the tests, accumulation of data bearing upon the specific objects contemplated could not be begun until 1924. Since then the tests originally scheduled have been completed and the data are available in standardized final form.

The results of the tests reported below were obtained with equipment specified by the cooperative committee. The tires used were standard equipment at the time of testing, and the results obtained apply specifically to these tires. They do not necessarily apply to tires that have been redesigned during or subsequent to the tests. This is particularly the case with reference to tires which have been specially designed for use in dual mountings, the profile heights being appreciably affected.

In applying the test data it should be remembered that an impact force, expressed in pound units, does not necessarily have the same effect upon materials as a static force of the same numerical value in pounds.

The term "cushioning effect" as used refers only to the vertical reaction between road and wheel, and is not to be confused with the popular term "riding quality," although it is somewhat akin to it. It should be borne in mind also that this research has been concerned with only one phase of certain economic problems, full consideration of which should give due weight to various other highway transportation factors relevant to the subject.

IMPORTANT CONCLUSIONS TO DATE

From the data at present available—and it should be borne in mind that they were obtained from the standard equipment used at the time of the tests—certain preliminary deductions may be drawn, namely:

1. Maximum impact forces obtained with motor truck tires in service can be measured with an accuracy sufficient for the needs of this investigation.
2. As static load increases, road impact reaction increases.
3. As static load increases, the ratio of road impact reaction to static load decreases.
4. Thickness and narrowness of tread rubber are desirable in reducing road impact reaction.
5. Increasing the thickness or profile height of rubber has a very marked effect in reducing road impact reaction in both single and dual mountings.
6. In the tire equipments tested, all of which were standard at the time of the tests, dual mounting caused heavier impact forces than the corresponding single mounting of the same total load-carrying capacity. (This was determined on a pneumatic-tired, 2-ton truck and a solid-tired, 5-ton truck.)

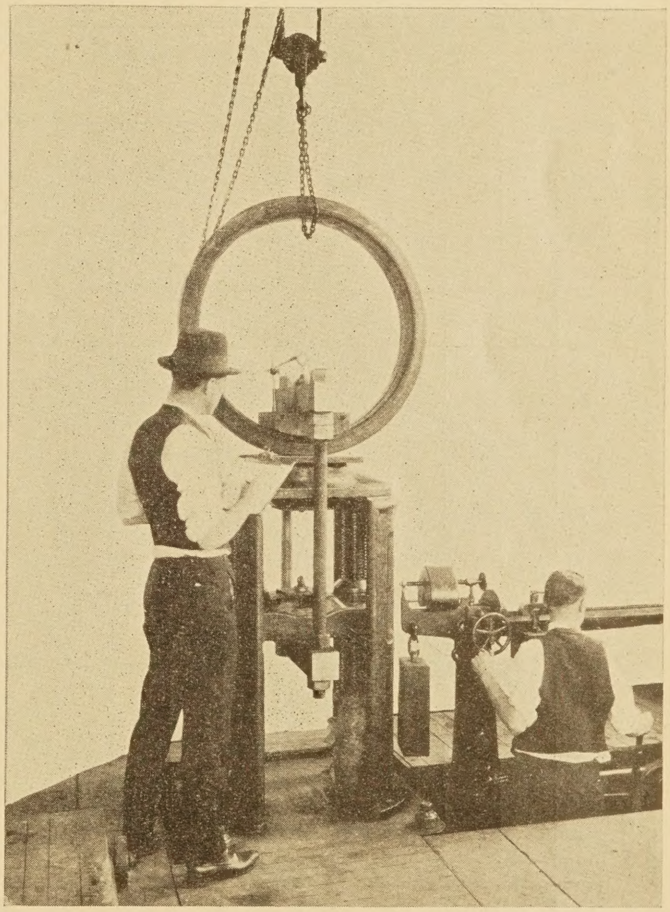


FIG. 1.—Method of mounting tire for static test

7. Appreciable variation of cross-sectional rubber, or breaks in its continuity, cause heavy repeated impacts to be delivered to the road.
8. Dual-mounted tires should always be mounted with the tread design staggered.

PROCEDURE OF THE TESTS

The tests of tire cushioning qualities are divided into two general groups, the static tests and measurements, and the impact tests. It is expected that, in most cases, a general relation will be found between these two test methods which will shorten and standardize

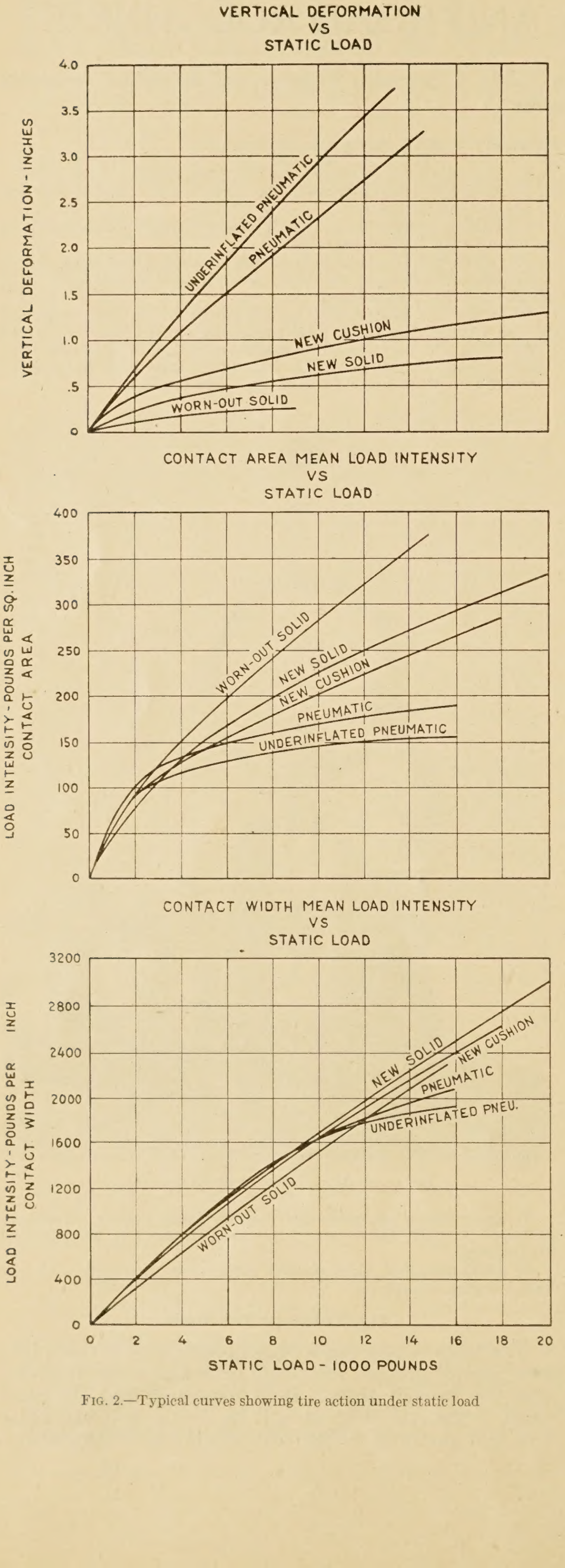


Fig. 2.—Typical curves showing tire action under static load

future test procedure. It may be that a relation of load versus work will be found which will yield a useful criterion in the correlation of the static and the impact tests.

Static tests.—In the static tests the tires were placed in a testing machine and vertical deflections were recorded as the load was slowly applied, from which data load *vs.* deformation curves were drawn. The areas of tire contact were taken at three loads on each tire, and reproductions were made of the impressions. From these the area, width, and length of contact were obtained at the three loads. The hardness of the tread rubber was measured by a Shore durometer. In addition the weights, dimensions, and trade names of the tires were recorded.

The method of mounting tires in the testing machine is shown in Figure 1; and typical curves indicating tire action under static loads are shown in Figure 2. These curves show the effect of load on vertical deformation, the mean intensity of load per unit of contact area, and the mean load intensity per unit of contact width. Figure 3 shows typical tire cross sections.

Impact tests.—In the impact tests the total vertical road reaction is determined by a specially constructed accelerometer mounted on the test trucks. This accelerometer was designed to measure the maximum vertical accelerations of the truck wheel and simultaneously to record the proportional deflections of the corresponding truck spring. The forces are computed by formula and may be resolved into sprung and unsprung components. The sprung component is determined by measuring the change of truck spring deflection from its static position at a given load to its position at the instant of impact. By referring to the proper portion of a previously prepared calibration curve for the truck spring, the change in spring pressure from that at static load is obtained. The unsprung component is determined by measuring the acceleration or deceleration of the truck wheel, adding thereto the acceleration of gravity and multiplying the sum by the known mass of the unsprung parts (i. e., the parts below and not supported by the truck springs).

The corresponding total force or road reaction is computed according to the formula:

$$F = m\ (a + g) + P,$$

wherein F = vertical road reaction in pounds,
m = mass of unsprung truck parts in poundals,
a = acceleration of unsprung truck parts in feet per second per second,
g = acceleration of gravity (32.2 feet per second per second),
P = truck spring pressure in pounds at instant of impact.

THE ACCELEROMETER

The accelerometer is mounted to follow faithfully the vertical movement of the right rear wheel of the truck by means of a tight, self-aligning bearing, and the instrument slides on guides on the truck body. The sensitive element is composed of a calibrated coil spring supporting a weight of known mass. A rod fastened to this weight acts as a guide and stylus holder. The stylus records the relative movement of the weight on sensitized paper which is rolled on a drum driven by the truck wheel, the paper moving approxi-

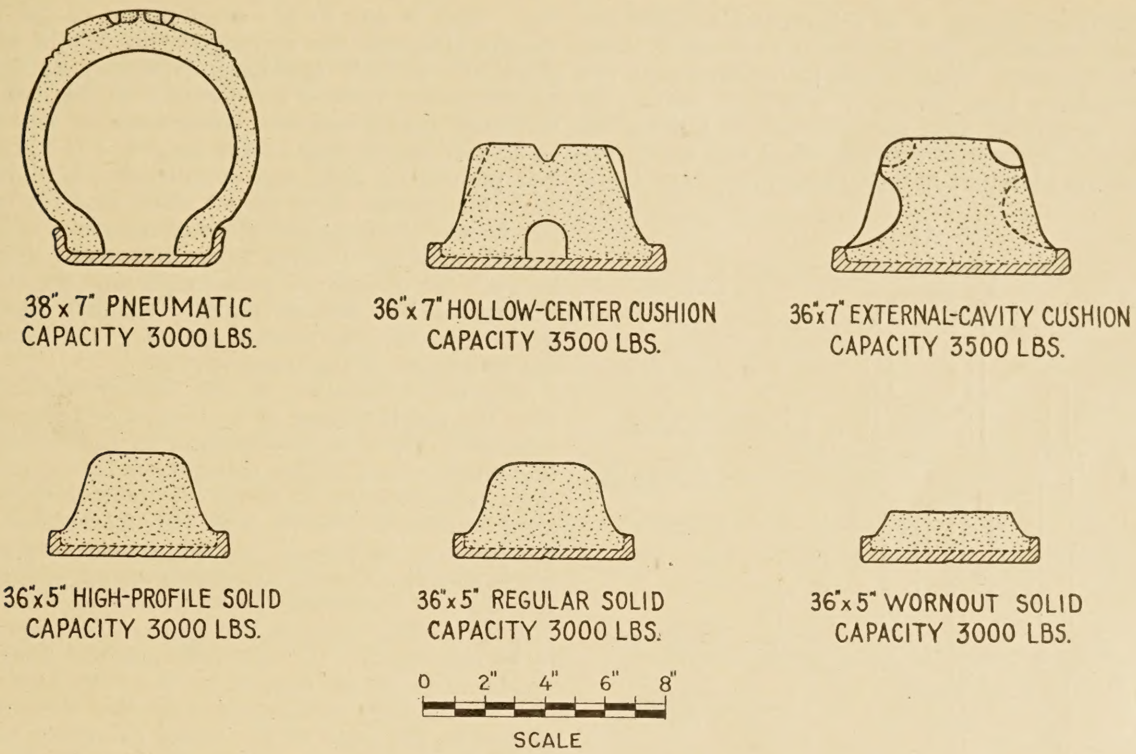


Fig. 3.—Typical tire cross sections

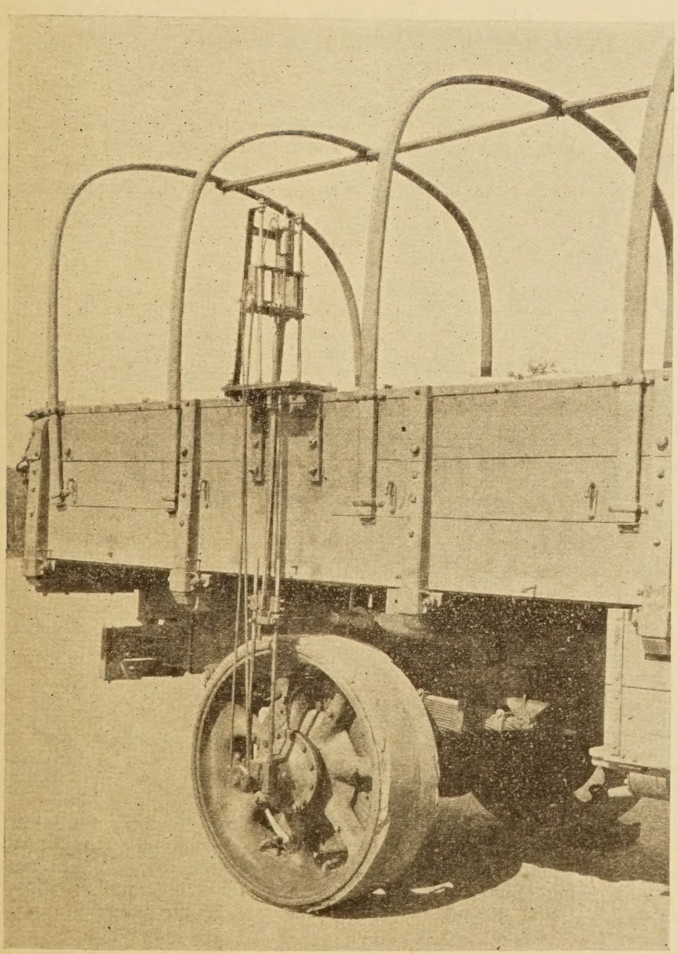


Fig. 4.—General view of instrument mounted on test truck

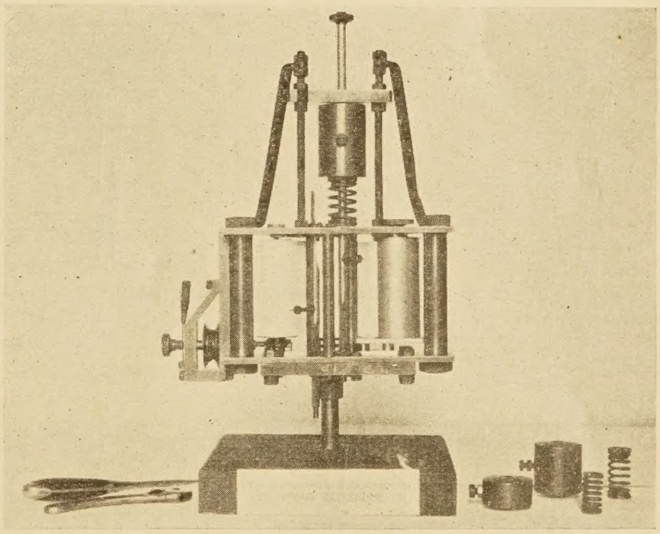


Fig. 5.—Detail view of instrument

mately 1 inch for each 45 feet of truck travel. A backstop is arranged above the accelerometer weight to prevent the upward movement of the weight beyond the position it assumes when the instrument is at rest and there is no initial compression on the spring other than that due to the weight itself. Figure 4 shows the complete apparatus installed on a truck. Figure 5 is a close-up view of the instrument proper, and the essential elements of the instrument are shown diagrammatically in Figure 6. Typical accelerometer records and the process of converting them into terms of force are shown for artificial obstructions in Figure 7 and for an actual road surface in Figure 8.

In Figure 7 the spring reading is recorded at zero for the condition of shock at each of the narrow obstructions because, as indicated by the instrument record, the truck spring is in substantial equilibrium immediately preceding the approach to each of these narrow obstructions. This is due to the fact that the obstructions have been placed on a smooth road. At the instant of impact with the obstruction there is an added reaction between the truck wheel and the pavement which may amount to many times the static loading. This is due to the acceleration of the unsprung weight although the sprung weight is still at its zero position. Strictly speaking, however, at the instant of maximum upward acceleration of the truck wheel, the truck spring has been compressed an amount equal to a small percentage of the height of the obstruction. For pneumatic tires this compression is about 10 per cent, for cushion tires 15 per cent, and for solid tires 20 per cent of the height of the obstruction. The spring record curve has sharper peaks downward than upward because the downward peaks represent the stopping of the wheel by the pavement, whereas the upward peaks represent the upward motion of the wheel against the resistance of the truck spring.

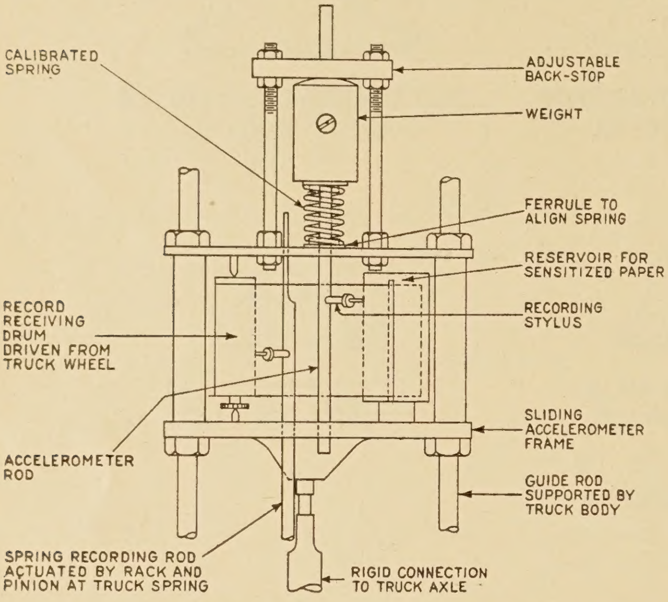
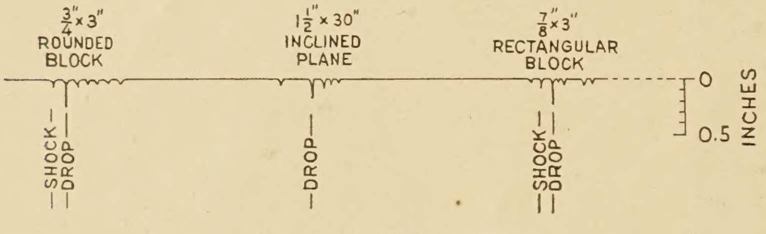


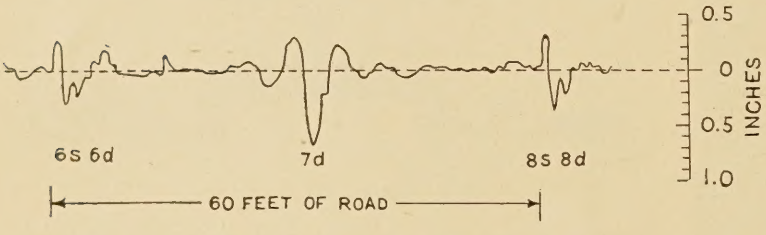
FIG. 6.—Diagram of the coil spring accelerometer

The characteristics of the accelerometer are such that the displacement of its weight as recorded on the sensitized paper is proportional to the acceleration of the truck wheel. The calibration factor depends on the characteristics of the accelerometer spring-weight combination and varies with the characteristics of the acceleration impressed on the accelerometer by the truck wheel. The accelerometer spring-weight characteristics are accurately determined and that combination is selected which is best suited to the accelerations to be measured. The characteristics of the impressed acceleration are influenced by the natural characteristics of the tire equipment and the unsprung truck weight in the case of the wheel dropping on a plane surface. In the case where the wheel passes over a relatively narrow obstruction at comparatively high speed, there may be an added correction dependent on the duration of this condition of shock.



ACCELEROMETER RECORD

The deviation of the accelerometer record from a straight line is a measure of the acceleration of the unsprung parts. This acceleration multiplied by the mass of the unsprung parts gives the force of the blow struck by the unsprung parts.



CORRESPONDING TRUCK SPRING RECORD

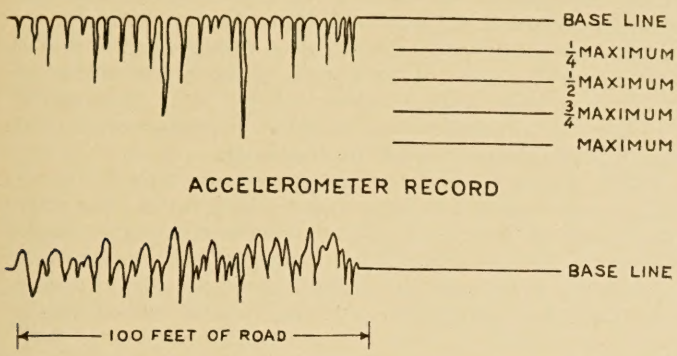
The truck spring under static conditions is deflected by the sprung load. At the instant of impact, the position of the truck spring above or below its average base line or static position is a measure of the sprung weight active at the instant of impact.

Test obstruction symbol	Unsprung weight				Sprung weight			Total vertical impact reaction	Total vertical impact reaction as a per cent of static load
	Accelerometer reading	Acceleration	Mass	Unsprung force	Spring reading	Variation from static	Sprung force		
	Inches	*Feet per second per second*	*Poundals*	*Pounds*	*Inches*	*Pounds*	*Pounds*	*Pounds*	*Per cent*
6s	0.08	162	22.5	4,370	0	0	1,625	5,995	255
6d	.25	506	22.5	12,110	.34	−1,605	20	12,130	516
7d	.18	364	22.5	8,915	.72	−3,400	−1,775	7,140	304
8s	.10	202	22.5	5,270	0	0	1,625	6,895	294
8d	.24	486	22.5	11,660	.40	−1,890	−265	11,395	485

The sum of the unsprung and sprung components is the total vertical road reaction. If this total impact reaction be divided by the total static reaction, it may be expressed as a percentage of the static load.

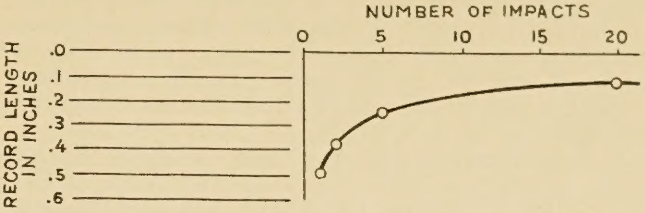
In the above example, the total static load per rear wheel is 2,349 pounds, of which 724 pounds is unsprung and 1,625 pounds is sprung. The accelerometer rate is 2,022 feet per second per second per inch of record, and 1 inch of truck spring record corresponds to a change of 4,720 pounds.

FIG. 7.—Analysis of typical record from artificial obstruction test



ACCELEROMETER RECORD

CORRESPONDING TRUCK SPRING RECORD



ACCELEROMETER ANALYSIS

The force of the wheel in striking the road is developed (a) by the blow of the parts below and not supported by the truck springs and (b) by the weight of the parts above and supported by the truck springs.

(a) The blow struck by the unsprung weight is found by multiplying its mass by its acceleration. The mass is obtained by weighing, and the acceleration from the above record to which the acceleration of gravity is added.

(b) The sprung weight is obtained by weighing.

Spring analysis

The average position of the truck spring is the same as when the truck is standing still, and the average pressure transmitted through the spring is equal to the weight of the body and load carried by each rear wheel.

For a wheel load of 9,000 pounds on a certain truck, the sprung weight was 7,800 pounds and the unsprung weight was 1,200 pounds, from which the unsprung mass was computed to be 37.3. The accelerometer used had a rate of 1,040 feet per second per second per inch.

$$F=m(a+g)+P$$

Number of impacts	Accelerometer reading	Acceleration (a)	$(a+g)$	Mass (m)	$m(a+g)$	Spring pressure (P)	Total impact force (F)
	Inches	*Feet per second per second*	*Feet per second per sec.*	*Poundals*	*Pounds*	*Pounds*	*Pounds*
1	.50	520	552	37.3	20,600	7,800	28,400
5	.25	260	292	37.3	10,900	7,800	18,700
10	.18	187	219	37.3	8,200	7,800	16,000
20	.12	125	157	37.3	5,900	7,800	13,700

FIG. 8.—Analysis of typical record from highway test

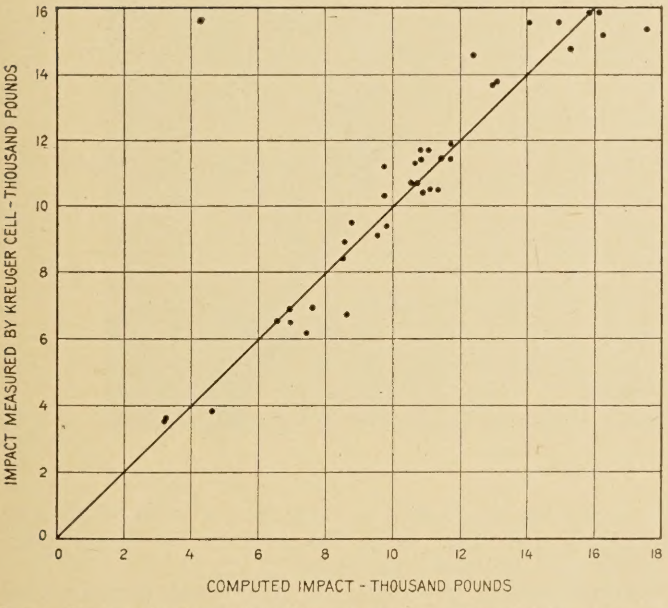


FIG. 9.—Diagram showing relation of impact reactions as computed from measured accelerations and as determined directly by means of the Kreuger cell

Calibration factors for various accelerometer spring-weight combinations have been determined for use with the full range of tire equipments included in the tests. These factors have been established analytically, indirectly, and directly. Formulæ, substantiated by experimental data, have been developed for the determination and checking of these calibration factors. The instruments have been calibrated by relating the instrumental records of impressed accelerations with determinations of the same accelerations from analyses of space-time records. That method of space-time analysis which assumes a parabolic shape of the original space-time curve over an appreciable period was selected as representing the average constant maximum acceleration. Should the instantaneous maximum be desired, it can be obtained from the calibration data already on hand and will vary from 100 to about 115 per cent of the parabolic maximum, according to conditions within the test range.

The accelerations were measured under two general conditions described by the terms "shock" and "drop." The condition of shock is defined as the reaction between the truck wheel and the pavement when the wheel is caused to pass over an elevation in or obstruction on the pavement. The condition of drop is defined as the reaction occurring when the wheel, falling under the combined influence of gravity and the truck spring, is suddenly stopped by the pavement.

On the above basis, for the test conditions to date, the errors in the measurement of acceleration will generally be within 10 per cent for all conditions of drop and for conditions of shock up to 10 miles per hour. For conditions of shock between 10 and 15 miles per hour, the error may be 15 per cent, and between 15 and 20 miles per hour for the condition of shock it may be 20 per cent. These values are considered conservative and are based on calibrations made by a special accelerometer subcommittee, which covered adequate ranges of speed and obstruction with pneumatic and new solid tires. The complete procedure and results of the accelerometer calibration are to be made the subject of a separate paper. Reference is made to an article in the December, 1924, issue of Public Roads by L. W. Teller, entitled "Accurate Accelerometers Developed by the Bureau of Public Roads."

After the accelerometer had been developed by the bureau and before it was used in the tests, it was checked by measuring the road reaction directly and comparing it with the computed reaction. The direct measurement was made by means of a Kreuger cell[1] placed in a pit so as to be squarely under the truck wheel at the instant of impact. The results of this check test are shown in Figure 9.


Fig. 10.—The smooth concrete test section

TIRES, SPRINGS, TRUCK SIZES, AND SPEEDS

The tires used were furnished by seven American manufacturers. The types were pneumatic, cushion, heavy-duty cushion, and high-profile, nonskid and regular solid tires. Tests have been made with new and worn-out tires, and with new tires cut to various heights. The pneumatic tires were tested at standard and three-fourths standard air pressures. The sizes varied from 32 by 3½ to 40 by 14, and the rated carrying capacities varied from 1,300 to 11,200 pounds per tire. They were used in single, dual, and tandem mountings, and as overloaded and oversized equipment.

The trucks used represent four American makes, the capacities of which are 1, 2, 3, and 5 tons. They were loaded up to 150 per cent of the carrying capacity of the standard types and sizes of tires with which they were equipped, using for the purpose 100-pound weights which were placed within the truck body in a definite manner and according to accepted practice in load distribution. The wheel loads were measured with portable road scales, which were frequently calibrated. In addition, a number of loads were sent to accurate platform scales for checking.

The trucks were driven at speeds varying by increments of 3 miles per hour from a minimum of 3 miles per hour up to the maximum speed obtainable under the load and test conditions. Speeds were determined by timing measured runs with a stop watch, and were checked for variation by the truck speedometer. Any run which showed a variation of more than one-half mile per hour from the average speed was discarded and the test was rerun.

Before use in the tests the rear truck springs were removed, cleaned, greased, calibrated, provided with jackets, and replaced. For reference, weight distributions of the empty trucks, truck measurements, and specifications were recorded.

ROAD SURFACES AND ARTIFICIAL OBSTRUCTIONS

In order to include actual road conditions, two 500-foot lengths of near-by highways were selected as test-road sections to represent extremes likely to be encountered by commercial traffic. One of these is typical of smooth concrete construction, and the other is an old stone-block pavement. A 16-wheel profilometer showed an average cumulative vertical varia-

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[1] For a description of the Kreuger cell and its use in measuring impact reactions see Public Roads, vol. 5, No. 10, Dec., 1924.

tion of 2.5 inches per 100 feet on the smooth concrete section and of 18.2 inches per 100 feet on the rough stone-block section. Both sections are straight and substantially level, and have only slight crowns. In general, the tests on these two sections were confined to tire capacity and 1½ tire capacity loads at a representative speed of 12 miles per hour. The results of these tests are expressed in terms of the number and magnitude of the impacts (i. e., vertical road reactions) produced as determined from the accelerometer records. A photograph of the stone-block pavement appears on the cover of this issue of PUBLIC ROADS, and the concrete road is shown in Figure 10.

To provide for duplication of test conditions, a very smooth, straight, and substantially level concrete road was specially built, and on it various steel obstructions were set in grout and secured by bolts. These obstructions included 30-inch inclined planes with rises of $\frac{13}{16}$, 1½, and 1$\frac{15}{16}$ inches, respectively; rectangular blocks 3 inches wide and with heights of $\frac{9}{16}$, ⅞, and 1⅛ inches, respectively, and rounded (segmental) blocks having 3-inch chords or bases and heights of ¾ and 1½ inches, respectively. These obstructions were about 4 feet wide and were placed at about 30-foot intervals along a guide line painted on the road. Figure 11 shows typical obstructions.[2]

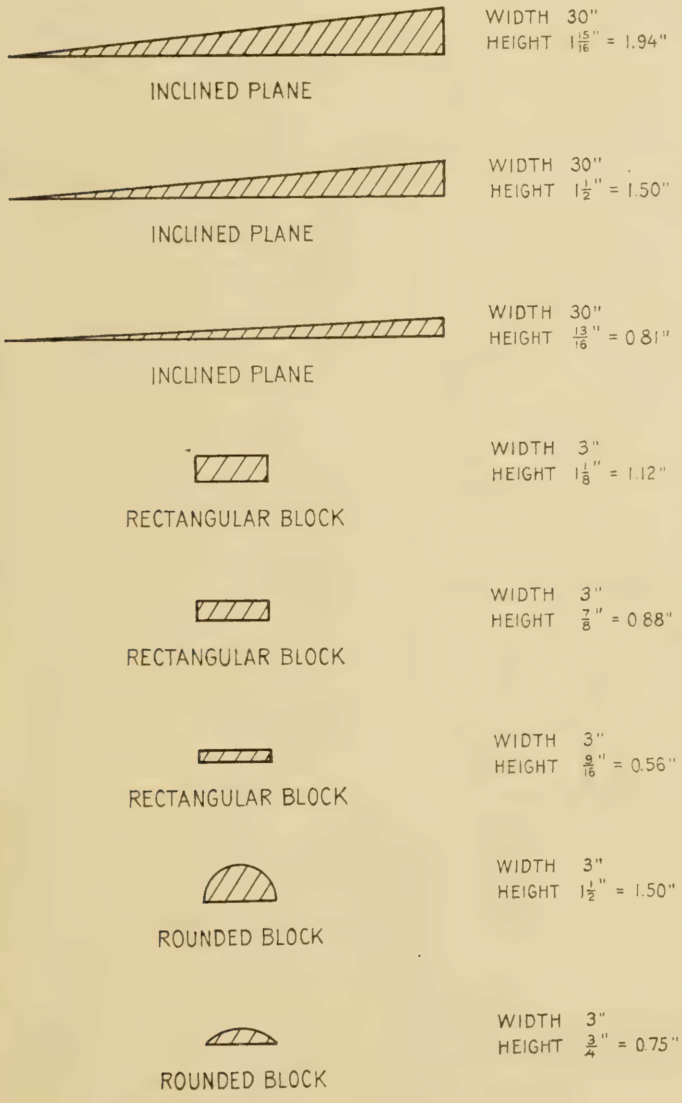

FIG. 11.—Typical obstructions used in the tests

An almost unlimited number of comparisons may be made from the test data showing the influence of the several variables on vertical impact reaction. It is not possible in this report to present exhaustive data with reference to all conditions studied; but general comparisons are made in the curves presented in Figures 12 to 31, and the original data in plotted form and the standardized computations are accessible to those who have a proper interest in making detailed studies or comparisons. In presenting this report the data are submitted as found, and the curves faithfully follow their plotted points. The ordinates of all the comparative curves represent the impact force expressed in terms of percentage of static load; to convert them into pounds it is necessary to multiply them by the corresponding static wheel loads in pounds according to Table 1. The overloaded and oversized tire tests were made at loads corresponding to 1½ the capacity of the rated sizes of tires.

TABLE 1.—*Wheel loads and tire sizes*

Tire type	Truck capacity	Wheel load capacity of rated size of tire		Tire size			
				Rated size		Over size	
		Single	Dual	Single	Dual	Single	Dual
		Pounds	*Pounds*	*Inches*	*Inches*	*Inches*	*Inches*
Pneumatic	2 tons	4,000	4,400	40 by 8	36 by 6		38 by 7
	3 tons		6,000		38 by 7		40 by 8
	5 tons		8,000		40 by 8		
Cushion	2 tons		3,400		36 by 5		36 by 7
	3 tons		7,000		36 by 7		36 by 8
	5 tons						
Solid	2 tons		4,000		36 by 4		36 by 6
	3 tons		6,000		36 by 5		36 by 7
	5 tons	8,400	8,400	40 by 12	40 by 6	40 by 14	40 by 8

EFFECT OF ROAD ROUGHNESS

Figures 12 and 13 show typical results obtained on the highway test sections. They show that pneumatic tires may make a rough road appear reasonably smooth and that solid tires in poor condition may make a smooth road appear unreasonably rough. Referring to the two figures we find, for instance, that with tires loaded to capacity and trucks operated at 12 miles an hour, the impact magnitude which occurs 50 times in 500 feet, when pneumatic tires are used, is 5 per cent greater than the static load on the smooth road and 40 per cent greater than the static load on the rough road. With worn-out solid tires on the two classes of road the impact occurring 50 times in 500 feet was 45 per cent greater than the static load on the smooth road and 270 per cent greater than the static load on the rough road. These curves also show that the forces developed on a road which is built and maintained smooth are, for given equipment, appreciably lower than the corresponding forces developed on a very rough road.

The impact road reaction of a motor truck depends in general on the four major variables—wheel load, truck speed, tire equipment, and road roughness or

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[2] No artificial ruts or pot holes have been included to date in the truck tests. A few tests have been made with the impact testing machine which is described and illustrated on page 13 of the November, 1924, issue of PUBLIC ROADS, by dropping the wheel upon a surface hollowed to a 40-inch radius. The results of the tests with this artificial rut did not seem to justify inclusion of such an obstruction in the truck tests.

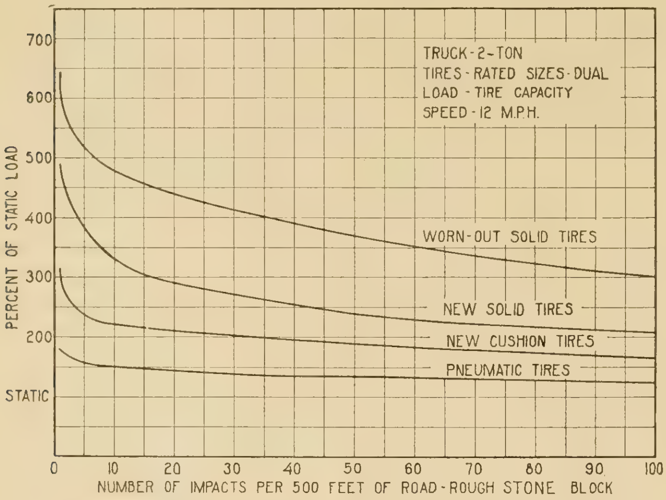

FIG. 12.—Influence of tires on vertical impact reactions on rough stone-block road

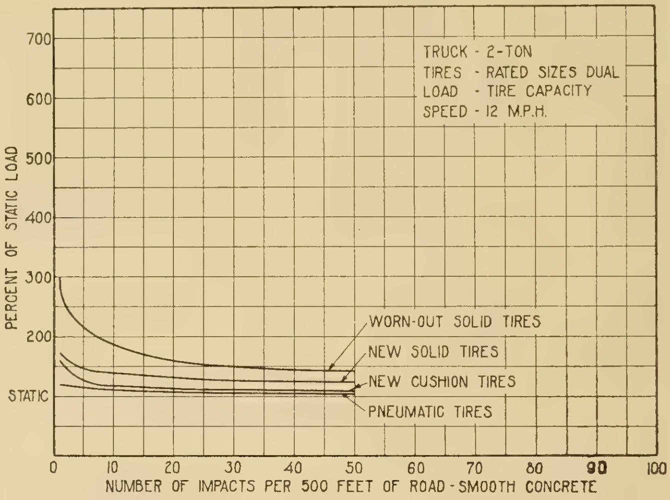

FIG. 13.—Influence of tires on vertical impact reactions on smooth concrete road

height of obstruction. The effects of other variables, such as the ratio of unsprung to sprung weights, truck spring flexibility, etc., are also felt to a greater or lesser degree, but manufacturing and operating conditions are generally such that the effect of these latter variables is not of major importance. The phenomenon of motor truck impact is separable into two distinct elements. There are two forces acting simultaneously and cumulatively on the pavement. One of these forces is the net truck spring pressure at the instant of impact, its magnitude varying with the spring deflection. The other is the force required to change the vertical velocity of the unsprung truck weight (that is, the parts below and not supported by the truck spring) as the wheel is vertically accelerated or decelerated by the pavement.

When the vehicle is standing still the road reaction is the static load on the wheel; but with the truck in motion this condition no longer obtains. What takes place when the wheel in motion strikes and passes over an obstruction on the road may be described in general terms as follows:

We will assume that the road is otherwise perfectly smooth and that the truck spring is at equilibrium in its normal position. As the wheel in its forward progress makes contact with the obstruction the tire is compressed and through its resistance to compression, varying according to the type of tire used, the unsprung weight of the truck is accelerated upward against the resistance of the truck spring reacting on the sprung weight of the truck body. The acceleration of the unsprung weight causes a reaction on the pavement the magnitude of which is measured by the product of the acceleration produced and the mass of the unsprung weight which is accelerated. Added to this reaction is the superimposed pressure of the truck spring which at this condition, called shock, is substantially that of the static sprung weight of the truck. The truck wheel then leaves, or tends to leave, the pavement and compresses the truck spring. The truck body, in response to this pressure, rises slowly upward and the entire rear end of the truck is momentarily completely in the air. Very soon, however, the upward inertia of the unsprung weight is overcome by the truck spring and the wheel descends upon the pavement to be decelerated through the elastic medium of the tire. Now, at this point the truck body is usually considerably above its normal position, the springs being correspondingly extended. The springs in this extended position cause a lesser superimposed pressure than the static sprung weight of the truck to be added to the second impact reaction caused by the deceleration of the truck wheel by the pavement. It may even be the case that the truck spring is so far extended that the resultant spring pressure is negative and actually lessens the effect of the deceleration of the unsprung weight. On the other hand it is possible, in the case of a long jump after striking the obstruction, for the truck spring to have fully extended and be in the act of closing again when the wheel strikes the road so that the sprung component may be even greater than the static spring pressure of the truck.

EFFECTS OF HEIGHT OF OBSTRUCTION, SPEED AND LOAD

In Figure 14 it will be noted that there is apparently a critical height obstruction, varying between 1 and 2 inches, according to the type of obstruction, at which the impact reaction approaches one of its maxima for the given tire equipment. The influences of the sprung to unsprung weight ratio, the truck load, and the truck spring characteristics are probably responsible for this condition.

Figures 14, 15, and 16 show typical results obtained from the artificial obstruction tests. The effects of three types of obstruction as influenced by height, and the influences of the speed of and load on the truck when passing over representative obstructions of these types are shown for typical tire conditions. It will be noted from Figure 14 that pneumatic tires did not yield reactions (under these test conditions) as great as two times the static load for any height of the inclined plane or for the 3-inch rectangular blocks, but that when worn-out solid tires were used, an impact reaction twice as great as the static load was caused by about 0.15 inch height for all three types of construction. A glance at this figure also shows that the impact reactions with worn-out solid tires may, under certain conditions, reach 11 times the static load; but that new solid tires show only about one-half of this increase over the static load.

In Figure 15 the influence of truck speed is shown. There is usually a critical speed between 12 and 15 miles per hour, at which, under the test conditions, the impact reaction will be at one of its maxima, particu-

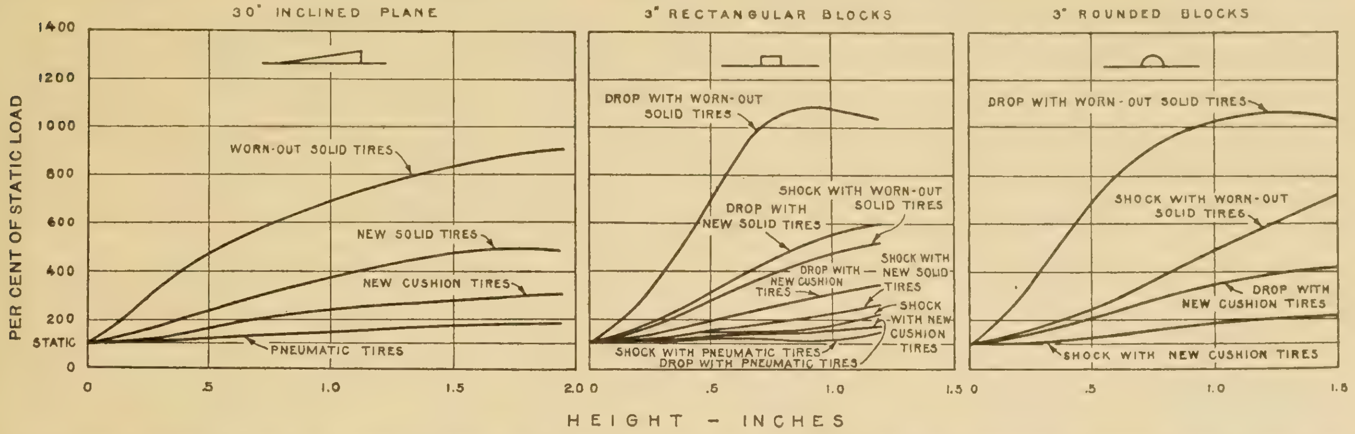

FIG. 14.—Effect of height and type of obstruction on vertical impact reaction. The tests were made with a 2-ton truck, equipped with rated-size dual tires, loaded to tire capacity, and operated at a speed of 12 miles per hour

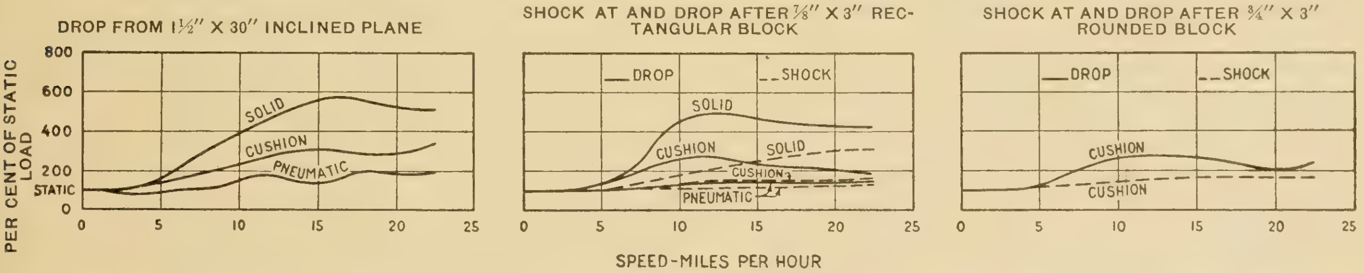

FIG. 15.—Effect of type of obstruction on vertical impact reaction as influenced by truck speed. These tests also were made with a 2-ton truck equipped with rated-size dual tires and loaded to tire capacity; and the truck was operated at various speeds

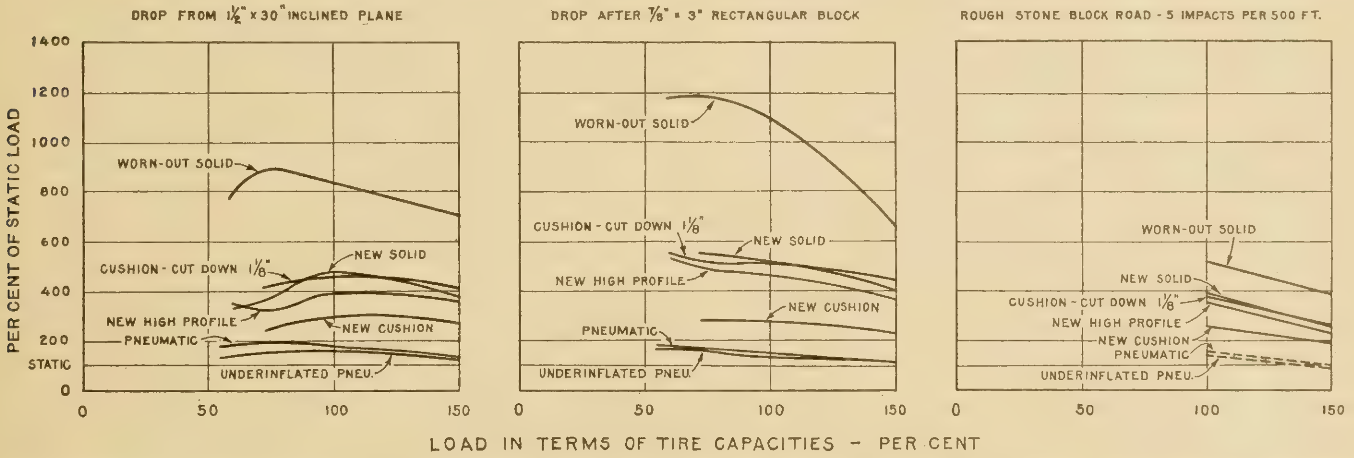

FIG. 16.—Effect of load in terms of tire capacity on vertical impact reaction with several types of tires and three conditions of impact. The curves represent tests made with a 2-ton truck equipped with rated-size dual tires, and operated at 12 miles per hour

larly for drop conditions. The reason for this is again found in the truck and load characteristics.

The most striking feature of the curves presented in Figure 16, which shows for various tires the effect of the load in relation to tire capacity, is the reduction in the impact with respect to the static load when the rated capacity of the tires is approached and exceeded. This phenomenon is probably due to the closer adherence of the truck wheel to the road under heavy load and also to the enforced cushioning of a greater part of the impact by the tires.

A clearer conception of the nature of the phenomena indicated by all three of these figures (figs. 14, 15, and 16) may be obtained by considering the behavior of two imaginary trucks; the first a truck of which practically the entire wheel load is derived from the unsprung weight and the body of which is but a skeleton supported by comparatively light springs. If, under such conditions, the tires were grossly overloaded, it will readily be seen that as long as there were any appreciable cushioning material present in the tires the excessive load directly on the tires would cause them, under all but the most extreme conditions of speed and road roughness, to remain in constant, intimate contact with the road. Comparatively slight obstructions would be embedded in the tire under the influence of the great load and there would be practically no difference between the cushioning qualities or springiness of various tires so long as they were able to carry the load at all. Now let us take another imaginary case in which the unsprung weight is a mere skeleton and the sprung weight, supported by adequate springs, is the principal load. An excessive wheel load, under these conditions, would not hold the tires so closely in contact with the road. The comparatively light wheel would be kicked upward as obstructions were encountered, the measure of the kick being determined by the cushioning properties of the tire. Having been raised into the air the wheel would, then, be returned to the pavement by the influence of the truck spring to be decelerated through the medium of whatever cushioning materials were in the tires. A truck under service conditions is a fortunate medium between the two imaginary extremes just described, but certain test conditions may be created, as shown by the figures, in which to a degree the effects described will be perceptible.

It is generally apparent from Figure 16 that as the static load increases the road impact reaction increases, but that the ratio of the impact reaction to the static load decreases.

EFFECTS OF TIRE EQUIPMENT

The next group of figures is concerned with tire equipments. Figures 17, 18, and 19 disclose the effects of the several variables on dual and single tire mountings, and Figures 20 and 21 show the relations between overloaded and oversized tire equipments. From an inspection of the three figures in the first group it is at once evident that the dual mounting of two smaller tires having approximately the same total carrying capacity as a larger tire mounted singly, causes a greater impact pressure on the road for a given condition. In the case of solid tires this effect of mounting may be such that the impact produced with dual tires will represent twice as great an increase over the static load as that produced with a single tire. The reason for this difference in impact forces caused by the two mountings is found in the cross sections of both the solid and pneumatic tires. The single tire, in each case, offers a thicker and narrower cushioning medium than the dual mounting. Consequently the acceleration or deceleration of the truck wheel is accomplished by the dual tire by less compression of the tire and therefore in a shorter time than a single tire. The acceleration or deceleration occurring in shorter time must necessarily be greater to produce the necessary change in vertical velocity for a given test condition. The acceleration of the unsprung weight being greater, the unsprung component of the road reaction is necessarily greater because it is the product of the acceleration times the mass of the unsprung weight. Given two otherwise equal cushioning media, that one which can effect the necessary change of velocity in the longer time interval will cause the lower impact force. These considerations lead to conclusions that:

1. Thickness and narrowness of tread rubber are desirable in reducing road impact reaction.
2. Increasing the thickness or profile height of rubber has a very marked effect in reducing road impact reaction in both single and dual mountings; and
3. In the tire equipments tested, all of which were standard at the time of the tests, dual mounting caused heavier impact forces than the corresponding single mounting of the same total load-carrying capacity.

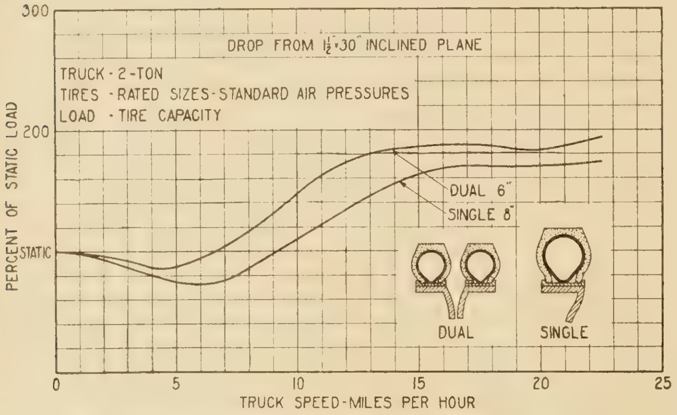

FIG. 17.—Single *vs.* dual tire mounting as influenced by truck speed

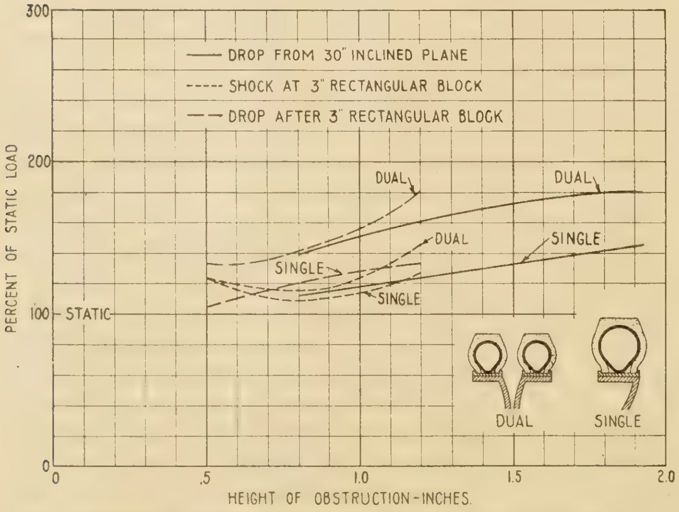

FIG. 18.—Single *vs.* dual tire mounting as influenced by height of obstruction

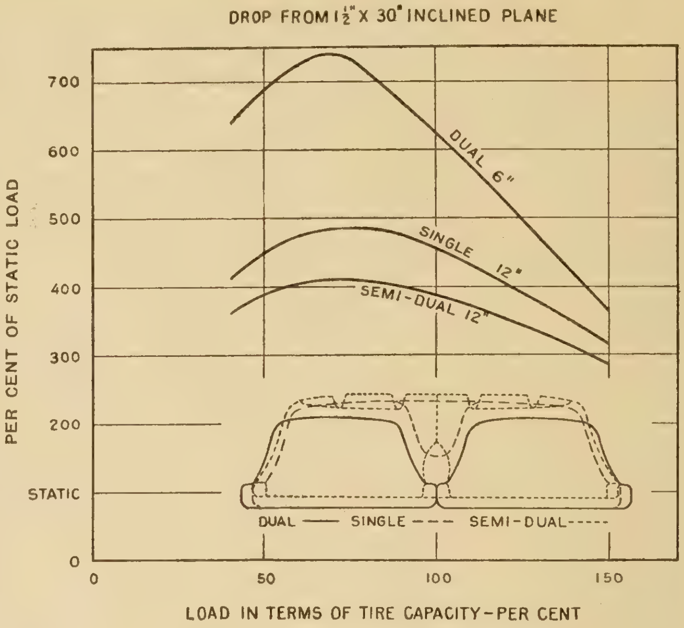

FIG. 19.—Single *vs.* dual tire mounting as influenced by wheel load. The tests were made with a 5-ton truck equipped with rated-sizes of tires and operated at a speed of 12 miles per hour

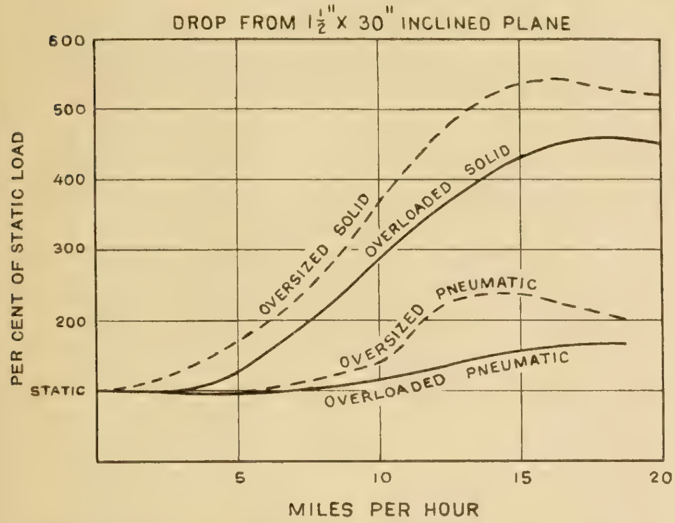

FIG. 20.—Effect of tire size on impact reactions resulting from drops from the 1½ by 30-inch inclined plane, using a 2-ton truck operated at various speeds. The overloaded tires carried a load equal to 150 per cent of their capacity, and the oversized tires carried the same load as the corresponding overloaded tires

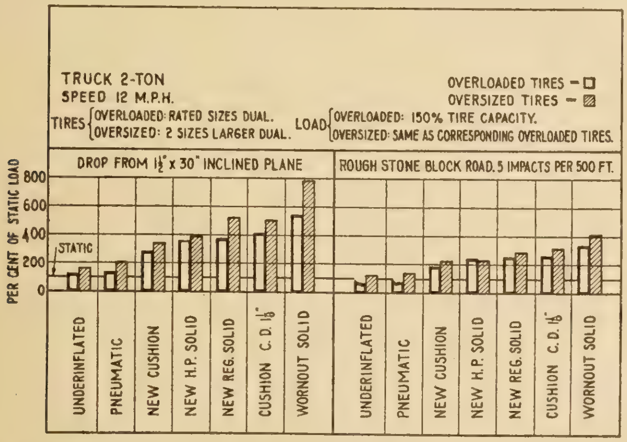

FIG. 21.—Relation between vertical impact reactions of overloaded and oversized tire equipments and the influence of tire type

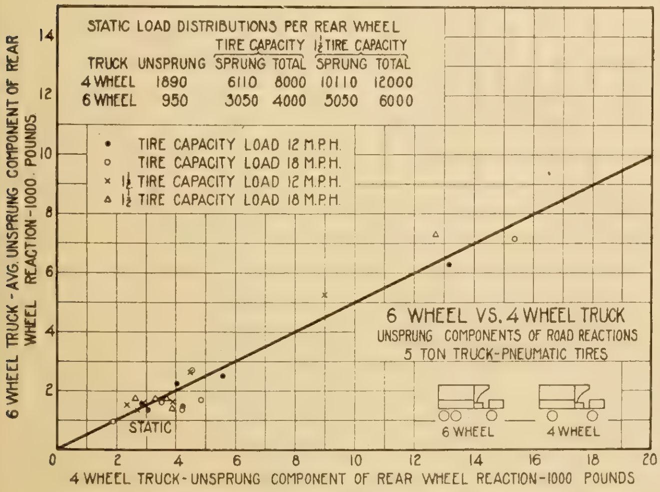

FIG. 22.—Impact relation of 6-wheel and 4-wheel trucks

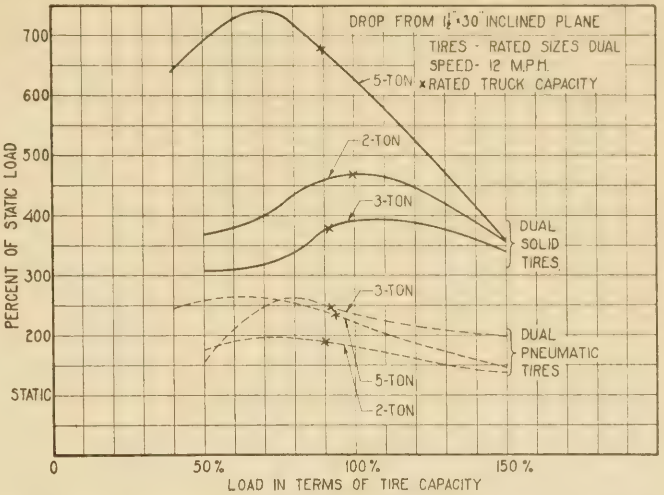

FIG. 23.—Vertical impact reaction as influenced by truck capacity and wheel load

Various relations between overloaded and oversized tires are brought out in Figures 20 and 21. It is at once apparent that, for truck wheels carrying the same load, those which are equipped with oversized tires cause heavier road impact reactions than those equipped with the rated and smaller sizes of tires. The reason for this is found in the dimensions of the two sizes of tires. If the cross sections of the two sizes of tires were superimposed it would be found that they differ markedly in width and comparatively slightly in height. The narrower tire would naturally be more easily deformed laterally, and because of this, it could effect the necessary change in velocity by being compressed a greater distance more slowly than the wider tire would be during a relatively short compression distance. As explained in the discussion of the dual and single mountings, the tire which decelerates the truck wheel from a given velocity comparatively slowly will cause a lower impact reaction on the road than the tire which necessitates a short, quick reaction. It is interesting to note, in Figure 21, that, the impact force is about 15 per cent greater than the static load for the rated size of underinflated pneumatic tires, and nearly 700 per cent greater for the worn-out solid tires, which were oversized equipment for the same truck and at approximately the same load. The relative actions of these overloaded and oversized tires supply further support for the conclusions with respect to the effect of tires stated above.

SIX AND FOUR WHEEL TRUCKS COMPARED

In order to determine the comparative effects of 6 and 4 wheel vehicles, tests were made with two 5-ton trucks which differed only in the rear-wheel arrangement. One was a standard 4-wheel truck equipped at the rear with dual pneumatic tires. The other was the same make and size of truck but was equipped with two rear axles which carried singly-mounted tires of the same cross section as those carried by the 4-wheel truck. Both rear axles were driven by the motor through tandem differentials and the distance

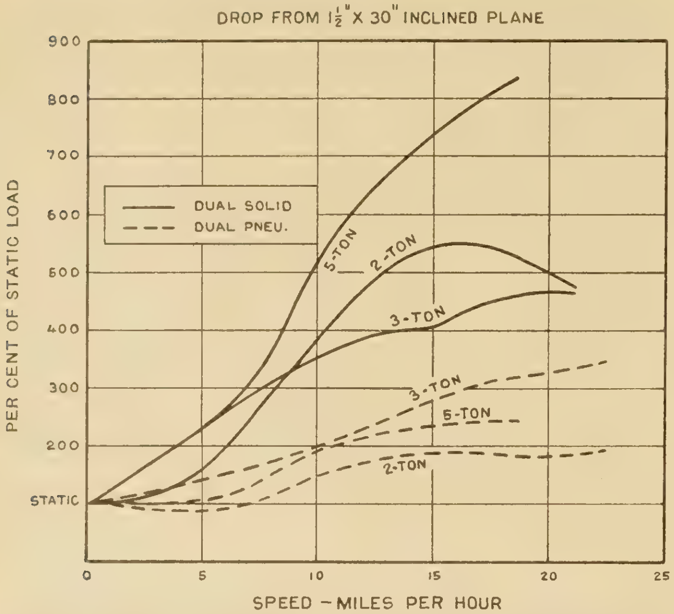

FIG. 24.—Effect of truck capacity on vertical impact reaction as influenced by truck speed. The trucks were equipped with tires of rated-size loaded to capacity

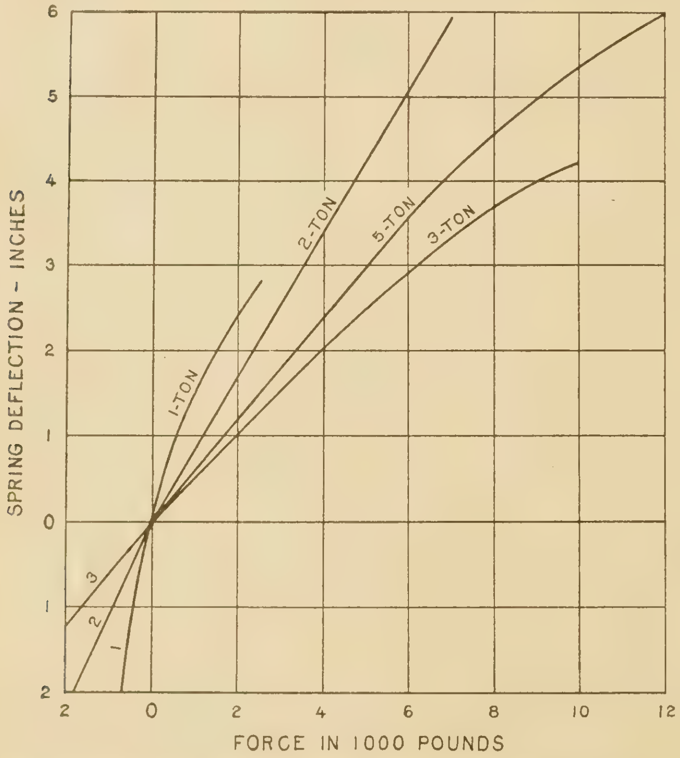

FIG. 25.—Truck-spring calibration curves, indicating the characteristics of the springs of the several test trucks

between axle centers was slightly over 4 feet. The usual truck spring was inverted, one spring being between the two axles on each side. The 6-wheel truck carried on its two rear axles a total load equal to that carried by the 4-wheel truck on its one rear axle.

In Figure 22, the unsprung components of the impact reactions of the 4-wheel truck are shown to be twice those of the equivalent 6-wheel truck, carrying the same load and operating under the same test conditions. This impact relation is particularly interesting in view of the results of other tests by the

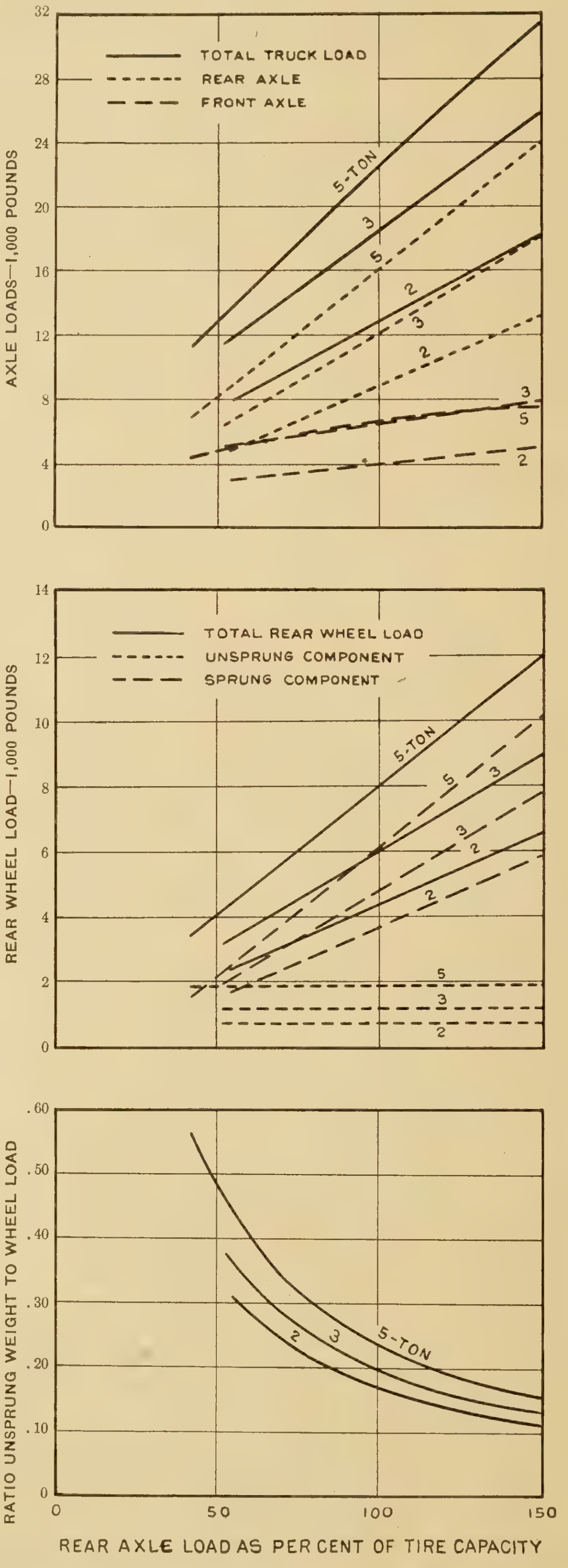

FIG. 26.—Diagrams showing distribution of the load of the various trucks equipped with rated sizes of dual pneumatic tires as used in the tests

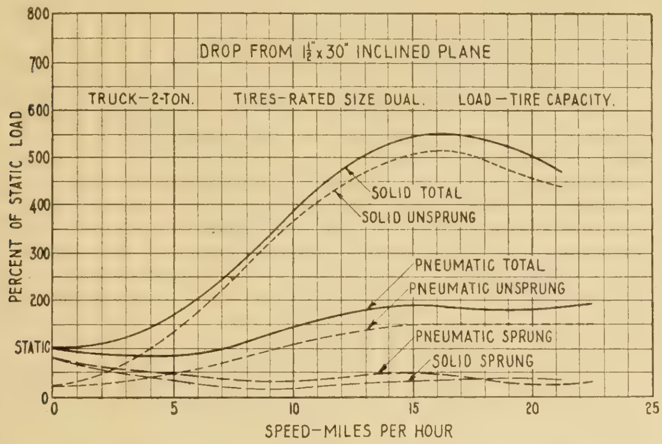

FIG. 27.—Components of the vertical impact reaction as influenced by truck speed

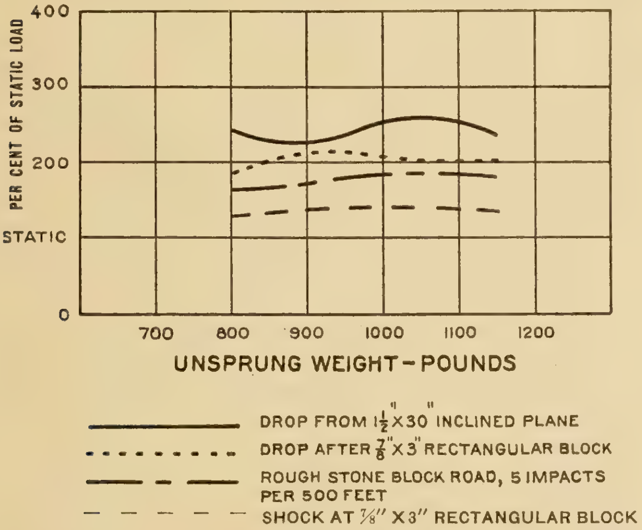

FIG. 28.—Curves showing the effect of varying the unsprung weight of a 2-ton truck equipped with new 5-inch dual cushion tires and operated with a rear-wheel load of 5,100 pounds at a speed of 12 miles per hour

bureau reported in the article entitled "The Six-Wheel Truck and the Pavement" which appeared in the October, 1925, issue of PUBLIC ROADS. These tests showed that the stresses in a concrete pavement caused by a 4-wheel truck are approximately twice as great as those caused by a 6-wheel truck carrying the same load.

Figures 23, 24, 25, and 26 are concerned with truck conditions. Figure 23 shows the effect of truck capacity and the influence of wheel load for the representative impact test conditions of the drop from a 1½ by 30 inch inclined plane at 12 miles per hour, with dual solid and dual pneumatic tire equipments. The rated truck capacities in terms of tire load are indicated and it will be seen that, with one exception, the trucks were overtired when equipped with sizes ordinarily specified as standard equipment, the trucks being fully loaded when the tires were carrying about 90 per cent of their rated capacities. It will also be noted that excessive overloading has a tendency to eliminate effects of individual truck characteristics because under the extreme sprung loads the truck springs are not permitted to function in a normal manner. As indicated in Figure 23, the 2, 3, and 5-ton trucks are about to seek a common curve to represent their behavior as the load increases for a given type of tire equipment. This does not mean that the reactions in pounds for each of these trucks are identical, but that the relative effects which are expressed as percentages of the static load are approximately the same. As previously discussed, these curves also show that excessive loads have a great leveling effect on cushioning qualities whether of the truck spring or tire equipment, and if the tires (with cushioning abilities between those of the new solid and the pneumatic types) were loaded to two and one-half or three times their rated carrying capacities it would appear that the increase over static load would become negligible for the given speed and obstruction conditions. The influences of the individual trucks and their spring characteristics are more pronounced in Figure 24, which shows the influence of the speed of the trucks. As tending to explain some of the apparently erratic indications of Figures 23 and 24 it should be stated that the truck which is designated as of 3 tons capacity was in reality a 5-ton chassis equipped with a 3-ton body. The characteristics of the springs are shown by the calibration curves in Figure 25 and the load distributions of the several trucks are given in Figure 26.

The vertical impact reaction separated into its components, caused by the sprung and unsprung parts is shown for a typical test condition in Figure 27. It has been found that the most severe impacts are obtained either at the shock on striking an obstruction or at the first rebound or drop after the obstruction has been passed. It is also true generally that the unsprung component of the road reaction is the major quantity, and, so long as the unsprung truck weight is decelerated (or accelerated) at all, it is likely to be the deciding factor in the total road reaction regardless of the compression of the truck spring.

The influence of the truck spring is felt, however, as shown in Figure 27, and under certain conditions, particularly at low speeds and with pneumatic tire equipment, the lessening of the sprung component due to the opening of the truck spring may be sufficient to cause the total vertical road reaction to be less than if the truck had been standing on the road.

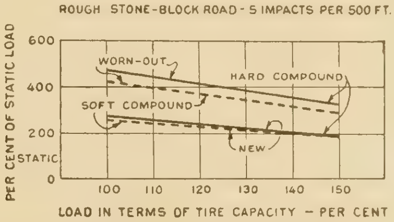

FIG. 29.—Effect of tire hardness on road reaction using new and worn-out tires

RESULTS OF AUXILIARY TESTS

The next group of figures shows results of a few auxiliary tests which have been made. Figure 28 shows the effect of varying the unsprung weight of a 2-ton truck equipped with 5-inch dual cushion tires and operated with a rear-wheel load of 5,100 pounds at a speed of 12 miles per hour. The unsprung weight was varied by lead weights affixed symmetrically to the rear axle by means of U bolts. Although this study is not exhaustive, it indicates that minor variations of the unsprung weight ratio do not seriously influence the road reactions. Figure 29 shows the effect of tire hardness or compound on the road reaction. A 3-ton truck was operated at 12 miles per hour with its rated-size, dual, solid-tire equipment. The tires were practically the same in cross section but varied in the hardness of rubber compound. They represent two standard makes and the tests indicate that the composition of tires in current use is not an important factor in impact reactions. Figure 30 shows the effect of internal tire temperature up to 190° F. for a typical test condition. A slight drop in the impact reaction is noted as the tire becomes warm. It was found to be impracticable to attain higher internal tire temperatures in the truck tests, and it is therefore not known what might occur under certain service conditions, wherein tires may reach their melting temperatures.

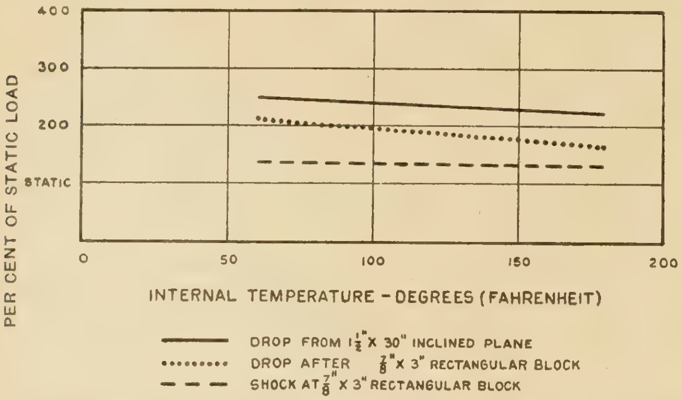

FIG. 30.—Effect of internal tire temperature on vertical impact reaction. The new tires used were 36 by 5 dual cushions. The truck was of 2 tons capacity and the load per rear wheel was 5,100 pounds. The unsprung weight per rear wheel was 1,116 pounds; and the speed was 12 miles per hour

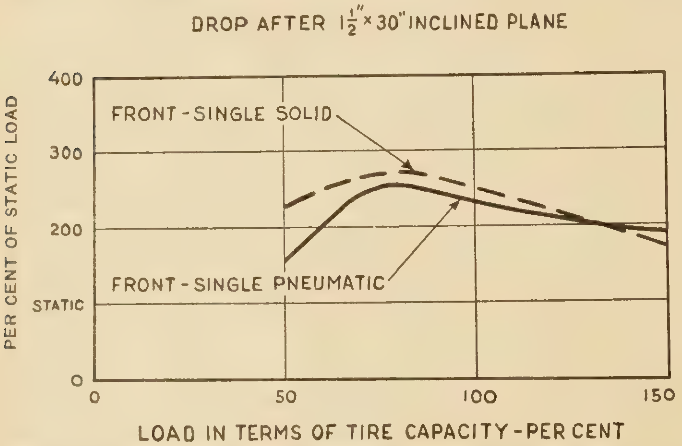

FIG. 31.—Effect on impact reaction of varying front-wheel tire equipment as influenced by the rear tire load. This test was made with a 3-ton truck, having rear wheels equipped with rated-size dual pneumatic tires, and operated at 12 miles per hour over the 1½ by 30-inch inclined plane

The influence of single pneumatic and single solid-tire equipments on the front wheel of a 3-ton truck the rear wheels of which were equipped with dual pneumatic tires is shown in Figure 31. The truck was operated at a speed of 12 miles per hour over the 1½ by 30 inch inclined plane. This study, though not exhaustive indicates that the front wheel equipment does not greatly influence the rear wheel reaction.

During the tests a certain type of new tire, which had a nonskid tread design now obsolete, was used on a truck as dual equipment. The tires were placed on the wheel with the tread designs opposite and then with the tread designs staggered. The markings on these tires were unusually deep and by varying the mountings as above an exceptional variation in cross-sectional rubber was obtained. This change in mounting was of itself sufficient to reduce the impact reaction from 3,600 to 1,200 pounds at each repetition of tread design (or about 8 inches) along a smooth concrete road.

From the tests made on these tires it is concluded that any appreciable variation of cross-sectional rubber or breaks in its continuity will cause heavy repeated impacts to be delivered to the road; and it is specifically recommended that dual tires should always be mounted with the tread designs staggered.

New French Magazine Appears

Revue Générale des Routes, to be Published Under the Auspices of the Ministry of Public Works

Announcement has been received of the publication in France of a new monthly magazine, the Revue Générale des Routes et de la Circulation Routière, under the auspices of the road office of the Ministry of Public Works.

Introduced by Senator Albert Mahieu, president of the permanent international association of road congresses, the new magazine has the backing of all groups interested in the solution of French road problems, including members of Parliament, the national administration, contractors, highway engineers, producers of road materials, and representatives of the motor vehicle industry.

The aim of the publication, as announced by M. Pierre Guieu, director general, is to contribute in all possible ways to the reconstruction of the French road system and to the improvement of methods of highway administration, financing, and construction.

In addition to the introduction by Senator Mahieu the first number contains interesting articles on grade crossings, slippery pavements, city street construction, and a technical index, including a bibliography, extracts from reviews, and a list of patents by recognized French authorities on the various subjects.

In subsequent numbers the discussion of slippery pavements and grade crossings will be continued; and among other articles promised is one on American roads by A. Antoine. Others will deal with the protection of dangerous crossings, studies of road materials, and the construction of forest roads. The forthcoming issues will also contain full information with regard to the Fifth International Road Congress at Milan, including programs and itineraries.

The offices of the new publication, of which M. Pierre Guieu is director general, are at 9 Rue Coëtlogon, Paris (VI). The foreign subscription rate is announced as from $5 to $6.

MAXIMUM STREAM FLOW

A FORMULA FOR GENERAL USE

By C. E. GRUNSKY, Consulting Engineer and Past-President American Society of Civil Engineer

THE FOLLOWING comments on maximum stream flow are prompted by the article published in PUBLIC ROADS, February, 1926, by Capt. C. S. Jarvis. This article is entitled "A General Formula for Waterways" and its author advocates the use of a formula for maximum stream flow based on area and a variable coefficient. He has selected the Myers formula in which maximum discharge is assumed to be proportional to the square root of the watershed area as best adapted to express the increase of discharge as area increases. He quotes in comparison formulas for the maximum rate of discharge as devised by Fanning, Talbot, and McMath.

All of these formulas have inherent defects and although applicable under certain conditions and in localities where data are available for determining the coefficients they obviously lack the elements which would make them generally applicable. The reasons why this must be so will be briefly set forth.

The intensity of the rainfall and its distribution throughout any given watershed is one of the main factors which determines the maximum rate of run-off at the outfall point of the watershed. But maximum rain intensity has a wide range throughout the world. It is largest on small areas. It decreases, in other words, as area increases. The run-off producing storm may, moreover, have a materially different effect if the storm travels downstream from that which it would produce if it traveled upstream.

The outline of the watershed, whether compact or elongated, broad at the base or narrow at the base, affects the rapidity of water concentration and therefore the peak run-off rate.

The gradient of the main lines of flow along which the water moves to arrive at the outfall determines the time which it takes the water from the various subdivisions of the watershed to get to the outfall point, and, therefore, the topographic and geographic features of the watershed must be regarded as having an effect upon the maximum rate of run-off.

Finally there is the effect of the size of the watershed on this rate. The maximum rate of rainfall, considered as an average over large areas, grows smaller as the areas in question grow larger; and in consequence the rate of run-off per unit of surface must likewise be smallest for the largest area.

It follows that every formula of the type of those which are noted by Captain Jarvis in his contribution are seriously at fault because they do not contain a rain-intensity factor. They are serviceable at the best only when the areas to which they are applied are similar in general topographic characteristics, in outline, in geologic surface structure, and in meteorological conditions. When this is the case, and then only, can such formulas be used, with some confidence, to approximate the maximum stream flow from watersheds of various areas, based always, however, on the known maximum stream flow from at least one area of similar characteristics which will serve to establish the value of the variable coefficient which appears in the formula.

GENERAL FORMULAS MUST INVOLVE RAINFALL INTENSITY

Any formula for general application must take into account the intensity of the rainfall which produces maximum stream discharge. This intensity when applied to an area of some extent, in contradistinction to a single point, decreases as the area increases. Furthermore the larger the areas of similar shape, the longer will it take the run-off to reach the outfall point. And, moreover, less time will be required for run-off waters to reach an outfall point from a fairly circular compact area than from an area which is much elongated.

It follows that the time which it takes the run-off to reach the outfall point from the various subdivisions of a watershed is an important element to be considered in devising a formula for the maximum stream discharge. Whenever it is possible to approximate the time (t) required for the maximum concentration of run-off water at the outfall point, it is possible also to compute from the known weather or meteorological conditions the volume of the precipitation in this concentration period, and this volume of water can then be made the starting point in the estimate of storm-water discharge. The further modifying factor depending on the absorptive properties of the surface of any area in question must also be taken into account and, because these surface conditions are variable, will necessarily appear in any formula as a variable. The effect of the retention of water in depressions or basins, such as lakes, swamps, and the like, when these are relatively small, will generally be sufficiently taken care of in the determination of the time to be allowed for run-off concentration. When a relatively large area of a watershed is lake or swamp surface, special consideration must be given to the effect of storage on the run-off and on the maximum stream flow, and ordinary formulas will no longer apply.

A rather complete analysis of this subject has been attempted by the writer, the results of which were contributed to the American Society of Civil Engineers.[1] It will not be necessary to review this analysis which showed the intimate relation of maximum stream discharge to maximum precipitation intensity. It will suffice to say that it led to the following suggestions:

1. The maximum rainfall intensity can, for all practical purposes, be expressed by the formula

$$I=\frac{C}{\sqrt{t}} \quad\text{(1)}$$

where I represents the maximum rain in inches per hour during the time t (in minutes), and C is a constant for any locality to be determined from the known maximum rate of rain in one hour, whenever small areas are involved, and from the maximum rain in some longer period, as, for example, 24 hours when large areas are involved. Thus, for example, if the maximum rainfall R, in one hour (60 minutes) is 1 inch then

$$R=\frac{C}{\sqrt{60}} \text{ and } C=\sqrt{60}, \text{ or } C=7.75.$$

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[1] The Sewer System of San Francisco, and a Solution of the Storm-water Flow Problem. Trans. Amer. Soc. of Civ. Engrs., Vol. LXV, p. 294; and Rainfall and Run-off Studies. Trans. Amer. Soc. of Civ. Engrs., Vol. LXXXV (1922), p. 66.

This is a value which will apply to some Pacific coast conditions. Or again if it be known that the maximum precipitation in 24 hours over some large area in question is 4 inches then

$$I_{24}=\frac{4}{24}=0.167 \text{ and } 0.167=\frac{C}{\sqrt{1,440}}, \quad \text{making } C=6.3$$

In the Middle West and on the Atlantic coast the values of the constant in the rainfall intensity formula will generally be three to five times as great for small areas and two to three times as great for large areas as on the Pacific slope.

2. The formula for maximum storm-water flow should take into account this maximum possible rain intensity and the topographic and soil characteristics of the watershed. Such a formula should, therefore, include as one of its elements rainfall intensity as deduced for the region in question from meteorological conditions, and the time of run-off concentration, and, also, a variable factor based on the soil characteristics of the watershed.

A GENERAL FORMULA RECOMMENDED

These considerations have led the writer to adopt and recommend a formula for maximum stream flow which is as simple as could well be desired and which will give dependable results whenever adequate data relating to rainfall and the surface characteristics of the watershed are known. The Jarvis modification of the Myers formula reduces it to a mere skeleton, making the maximum discharge proportional to the square root of the area, but retaining therein a factor p or P which the engineer must evaluate on some information relating to the same factors which enter into the type of formulas intended for more general application. The principal purpose of the Jarvis formula, and this only within certain limits, seems to be to approximate discharge from a watershed when the discharge from some other watershed of similar surface character and in the same general region is already known.

Other formulas such as the McMath formula and the Burkli-Ziegler formula[2] take rainfall intensity into account. They are in a different class from the Myers formula as modified by Captain Jarvis and such other formulas as those of Kuichling, Fuller, and Metcalf and Eddy in which the precipitation factor does not appear. But even the McMath and Burkli-Ziegler formulas have their limitations which it is not proposed to discuss at this time.

The formula which the writer suggests as one for general use is as follows:

$$D_m=413\,aMI \tag{2}$$

In this formula D_m = maximum rate of discharge from the watershed in second-feet.
M = watershed area in square miles.
a = a variable coefficient depending on the character of the surface of watershed.
I = the maximum average rainfall intensity throughout the entire watershed, expressed in inches per hour, during the time (t) in minutes, which is the time that it will take water under maximum rainfall conditions to concentrate at the outfall point and produce maximum discharge.

It is to be noted that the application of this formula to a part only of a watershed may give a result larger than that obtained under its application to the entire watershed. This is not inconsistent; but in that event the larger value of D_m, whichever it may be, will represent the desired maximum stream flow.

For use in the formula the value of I, the rain intensity which produces maximum discharge, is to be ascertained as already explained. The coefficient a which represents the effect of soil absorption must decrease as the time which it takes the water to reach the outfall point increases. For a small impervious area, $a=1$. These facts have suggested the following expression for a

$$a=\frac{60}{60+c\sqrt{t}} \tag{3}$$

The coefficient c in this equation is to be determined from the surface conditions of the watershed and the following values are tentatively suggested:

For impervious areas ---------------------- $c=0.1$
For mountainous areas -------------------- $c=2.0$
For rolling country ----------------------- $c=3.0$
For rolling and flat country (by interpolation) $c=(3 \text{ to } 10)$
For flat country (ordinary soil) -------------- $c=10.0$
For sandy regions ------------------------- $c=50.0$

These values are intended to apply under ordinary conditions in temperate climates. They are probably too small in localities where rain falls on frozen ground or on snow.

TABLE 1.—*Values of the coefficient, a, computed from the formula,* $a=\frac{60}{60+c\sqrt{t}}$, *for use in the formula* $D_m=413\,aMI$

Critical time (t) minutes	Impervious areas $c=0.10$	Mountainous areas $c=2$	Rolling country $c=3$	Flat country $c=10$	Sandy regions $c=50$
5	1.00	0.93	0.90	0.73	0.35
10	1.00	.90	.87	.66	.27
20	.99	.87	.82	.57	.21
30	.99	.85	.79	.52	.18
60	.99	.79	.72	.44	.13
120	.98	.73	.65	.35	.099
180	.98	.69	.60	.31	.082
600	.96	.55	.45	.20	.047
1,440	.94	.44	.35	.14	.031
7,200	.88	.26	.19	.066	.014
14,400	.83	.20	.14	.048	.010
28,800	.78	.15	.11	.034	.007

Wherever dependable information relating to maximum storm water discharge exists for one or more streams which are comparable in the matter of surface characteristics with those of some stream for which the maximum possible discharge is to be ascertained, then the proper value of a or of c can be determined with fair dependability.

It remains to be stated that the simple relation existing between I and R permits of a substitution of R for I in the formula which may therefore be written in two forms:

1. When I—that is, the maximum intensity of rain on the watershed during the critical time—t, has been ascertained then as already noted:

$$D_m=413\,aMI$$

2. When R—the maximum rainfall (in inches) in one hour, is known then, because $I=\frac{R\sqrt{60}}{\sqrt{t}}$, there will be:

$$D_m=\frac{3,200\,aMR}{\sqrt{t}} \tag{4}$$

APPLICATION OF FORMULAS ILLUSTRATED

The application of these formulas can best be illustrated by a few examples:

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[2] Rainfall and Run-off Studies. Trans. Amer. Soc. of Civ. Engrs., Vol. LXXXV, p. 100.

Example I.—What is the maximum discharge of a river draining a mountain watershed of 1,900 square miles, in which the maximum amount of rain averaged for the entire watershed is 8 inches in 24 hours, and for which the critical time, t, is 12 hours or 720 minutes?

Here $c=2$ and therefore from equation (3)

$$a=\frac{60}{60+2\sqrt{720}}=0.53$$

This value of a could have been taken from the table in the mountain column by interpolation.

Equation (1) gives

$$C=\frac{8}{24}\times\sqrt{1,440}=12.6$$

and

$$I=\frac{12.6}{\sqrt{720}}=0.47 \text{ inches per hour.}$$

Therefore from (2)

$$D_m=413\times0.53\times1,900\times0.47=195,000 \text{ second-feet}$$

or; because

$$R=\frac{12.6}{\sqrt{60}}=1.63.$$

Therefore from (4)

$$D_m=\frac{3,200\times0.53\times1,900\times1.63}{\sqrt{720}}=195,000 \text{ second-feet.}$$

The conditions suggested in this example are comparable with those which prevail in the watershed of the American River above Folsom, Calif., which from a watershed of 1,900 square miles has probably discharged 180,000 to 200,000 second-feet at the peak of a flood period (1861–62).

Example II.—What is the maximum discharge of a river draining a watershed three-fourths mountainous and one-fourth rolling or foothill land, 9,000 square miles in area, in which the maximum amount of rain in two days over the entire watershed is 6 inches and for which t=30 hours or 1,800 minutes?

Here a will be found by interpolation between two columns of the table or

$$a=\tfrac{3}{4}\left(\frac{60}{60+2\sqrt{1,800}}\right)+\tfrac{1}{4}\left(\frac{60}{60+3\sqrt{1,800}}\right)=0.39.$$

Equation (1) gives

$$C=\frac{6}{48}\sqrt{2,880}=6.7$$

and

$$I=\frac{6.7}{\sqrt{1,800}}=0.158 \text{ inches per hour.}$$

Therefore from (2)

$$D_m=413\times0.39\times9,000\times0.158=229,000 \text{ second-feet.}$$

The conditions suggested in this example are comparable with those which prevail in the watershed of the Sacramento River, Calif., at Red Bluff, where the maximum recorded discharge has been about 250,000 second-feet.

Example III.—What is the maximum discharge of a river draining a watershed of 250,000 square miles, equal parts of which are mountains and rolling country. in which the greatest amount of rain in one month over the entire watershed is 6 inches and for which the critical time, t, is 20 days or 28,800 minutes?

Here a is again to be found by interpolation between the two columns of the table or:

$$a=1/2\left(\frac{60}{60+2\sqrt{28,800}}\right)+1/2\left(\frac{60}{60+3\sqrt{28,800}}\right)=0.13$$

Equation (1) gives:

$$C=\frac{6}{720}\sqrt{43,200}=1.73$$

and

$$I=\frac{1.73}{\sqrt{28,800}}=0.0102$$

$$D_m=413\times0.13\times250,000\times0.0102=137,000 \text{ second-feet.}$$

This third illustration is typical of the conditions which prevail in the watershed of the Colorado River, which has an area of about 245,000 square miles (above Yuma) and from which the maximum run-off is at times about 0.6 second-foot per square mile.

When applied to small areas as in the case of studies for capacities of storm-water sewers, it will be found convenient to use the formula in its second form, equation (4), and to express area in acres instead of square miles as the unit. The formula will then appear in the form:

$$D_m=\frac{5a\ M'\ R}{\sqrt{t}} \quad \text{-----} (5)$$

Here M' is the area of the watershed in acres, R the maximum rainfall in one hour and a the coefficient determined by surface characteristics as shown in the table.

Example IV.—To illustrate the application of the formula to a small area, let it be required to determine the maximum storm water flow from a combined urban and suburban area 5,000 acres in extent of which one-third is impervious and two-thirds rolling country, and which is so located that the critical time t is 90 minutes in a region for which the maximum rainfall in one hour is known to be 3 inches.

In this case, using the table, the value of a will be:

$$a=\tfrac{1}{3}\times0.99+\tfrac{2}{3}\times0.69=0.79.$$

Equation (5), therefore, gives

$$D_m=\frac{5\times0.79\times5,000\times3}{\sqrt{90}}=6,200 \text{ second-feet.}$$

This is at the rate of about 800 second-feet per square mile.

In using any formula for maximum stream flow it must be remembered that the value of R as determined from single station records is not the value of R for an area of considerable extent, because the area over which there is coincident maximum rainfall is limited. For the large area this value will be somewhat less than that deduced for the time t from single station records. The formula may, therefore, be assumed to include a moderate factor of safety when, as is customary, the single station records are used without any correction for the lapse of time between the arrival of a storm in one part of the watershed and its arrival in another.

The hope is ventured that engineers who are concerned with the determination of maximum storm-water flow from any watershed can be made to realize that no formula should be considered generally applicable unless it includes two variables, one based on maximum precipitation and the other on the surface characteristics of the watershed. In the case of all other formulas their limitations should be clearly set forth. They should not be called generally applicable.

DISCUSSION

By **O. L. Grover**, Bridge Engineer, United States Bureau of Public Roads

The need for a general waterway formula is apparent when one is confronted by several formulas and tables which are in use in different sections of the country. A formula is an aid in checking and comparing sizes of waterways but is generally not sufficiently close for the results to be used without the field data which should be obtained so far as possible. No formula should be used to supplant reliable field information. Frequently rough values are wanted in a hurry and later the checking of tentative sizes which have been determined in other ways is of great value. A means of comparing sizes is needed. It is believed such a means is at hand in the published charts on which the Myers formula has been platted. When applied on topographical maps so that drainage areas and topographical features are before one, great benefits should be had and for a minimum of labor and time.

Formulas having more factors to represent more or less independent variables may have justifiable advantage in finding proper sizes for storm-water sewers and other expensive structures, but the comparatively simple method suggested by Mr. Jarvis has much to recommend it for open country stream crossings where complete and accurate surveys are the exception rather than the rule.

By **C. S. Jarvis**, Associate Highway Engineer, United States Bureau of Public Roads

The formula proposed and successfully used by Mr. Grunsky is a good example of refined methods applied to a very elusive problem. The results are dependable in proportion to the basic knowledge, experience, and judgment of the manipulator.

Any expression that is flexible enough to cover the entire field of design is also capable of yielding erroneous results, because of the wide range in coefficients. The safest basis of control seems to be that which gives a direct comparison, such as the percentage scale affords.

If the variations due to extent of watershed are accounted for by employing some definite function of the drainage area, whether it be the square root or any other, then the influence of slope, vegetation, soil porosity precipitation, and other similar influences maywell be measured by changes in the coefficient, if the modified Myers formula is well founded.

To test the Myers scale for practical purposes, 12 problems were submitted to engineers of the highway and agricultural engineering branches of the United States Bureau of Public Roads; of the Southern Railway Co., the United States Army, and the Geological Survey.

This set of problems was solved in from one to two hours by reference to topographic maps, rainfall records, and the tabulated data published in connection with the paper entitled "Flood Flow Characteristics" published in the Proceedings of the American Society of Civil Engineers, December, 1924, to December, 1925. The results were as follows:

Stream and location	Drainage area	Ratings on the Myers scale determined independently by five engineers					Authoritative solutions	
		No. 1	No. 2	No. 3	No. 4	No. 5	Rating	Required waterway area [1]
	Sq. miles	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*Sq. ft.*
Bear Creek, near Denver, Colo	-------	15.0	14	12	14.0	14	------	-------
Hinkson Creek, Columbia, Mo	-------	15.0	12	13	17.0	14	------	-------
Santa Fe Creek, Santa Fe, N. Mex	-------	6.0	10	7	7.0	8	------	-------
Catoma Creek, Montgomery, Ala	285	10.1	12	15	10.0	10	12.0 8.4	[2] 2,030 [3] 1,420
White River, Seattle, Wash	424	14.0	14	12	15.0	12	12.3 8.8	[2] 2,530 [3] 1,810
Rock Creek, near Gettysburg, Pa	-------	21.0	30	25	25.0	25	------	-------
Marais des Cygnes (Osage), Ottawa, Kans	1,247	14.1	12	17	18.0	13	16.6	[2] 5,860
Rouge River, Detroit, Mich	-------	3.0	3	4	3.0	4	------	-------
Tallahatchie River, Batesville, Miss	1,595	12.0	9	11	12.0	10	9.7	3,870
Kootenai River, Bonner's Ferry, Idaho	13,000	12.3	12	10	11.0	12	11.0	[2] 12,600
Klamath River, near its mouth, northwest Calif	15,500	30.0	25	20	25.0	15	20.0 14.5	[2] 25,000 [3] 18,200
Rio Grande, Brownsville, Tex	230,000	10.0	6	6	7.5	8	7.9	[2] 38,000

[1] Derived from fundamental formula, $w = 10\,p\sqrt{M}$, where M equals drainage area in square miles, w is the waterway area required and p the percentage rating.

[2] Rare.

[3] Frequent.

Only first approximations were attempted in each case, as the time was limited, but the results are in close enough agreement with each other and with the authoritative solutions, whenever available, to serve as a check.

The concensus of opinion among those who observed or participated in this test is that a single problem solved by other methods would ordinarily require nearly the same period of time as was used for the entire set with the Myers scale and method of approach.

Among the various rational and empirical waterway formulas which have met with favor, such as the one proposed by Mr. Grunsky, there seems to be the need of a common denominator or means of translation of results. Those who have tested the Myers scale have regarded it as a basis of comparison, where for example the flood intensities of drainage areas rating 10 per cent and 20 per cent respectively are shown in their correct relation even though the watersheds may be unequal in extent.

The adopted coefficient p for the modified Myers formula in any given case is a percentage representing the algebraic sum of all influences affecting run-off, and in this group rainfall, slope, and soil porosity are of prime importance.

Since the publication of the article "A General Formula for Waterways" in PUBLIC ROADS, February, 1926, two diagrams have been prepared for use in determining the Myers scale rating of a stream where the drainage area and the discharge are known. In the diagram on page 87 the ordinate q represents the run-off per square mile while the diagram on page 88 employs Q, the total discharge in cubic feet per second corresponding to the observed flood stage. These diagrams are intended as an appendix to the former article.

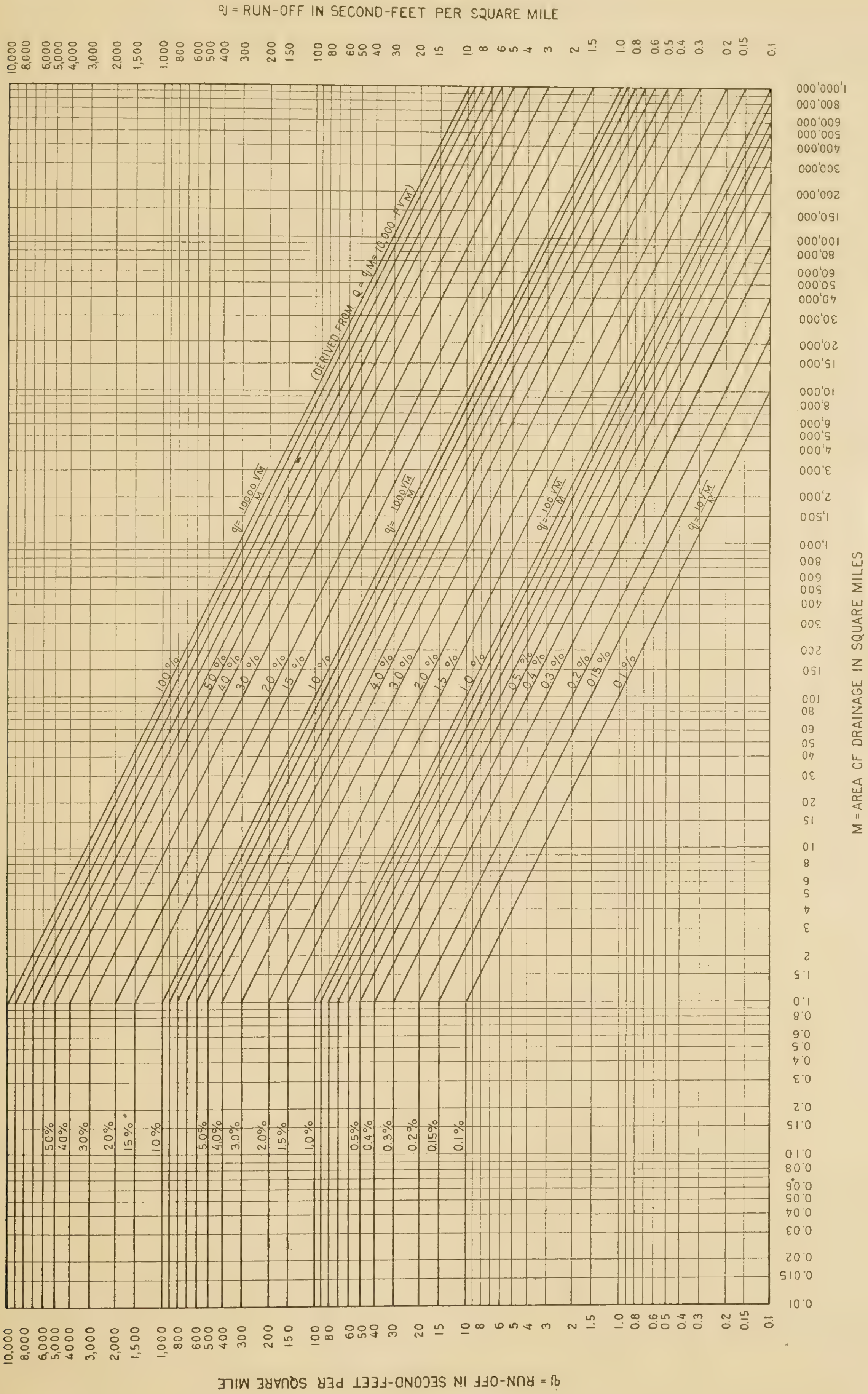

Diagram showing percentage rating on the Myers scale for any given yield per square mile

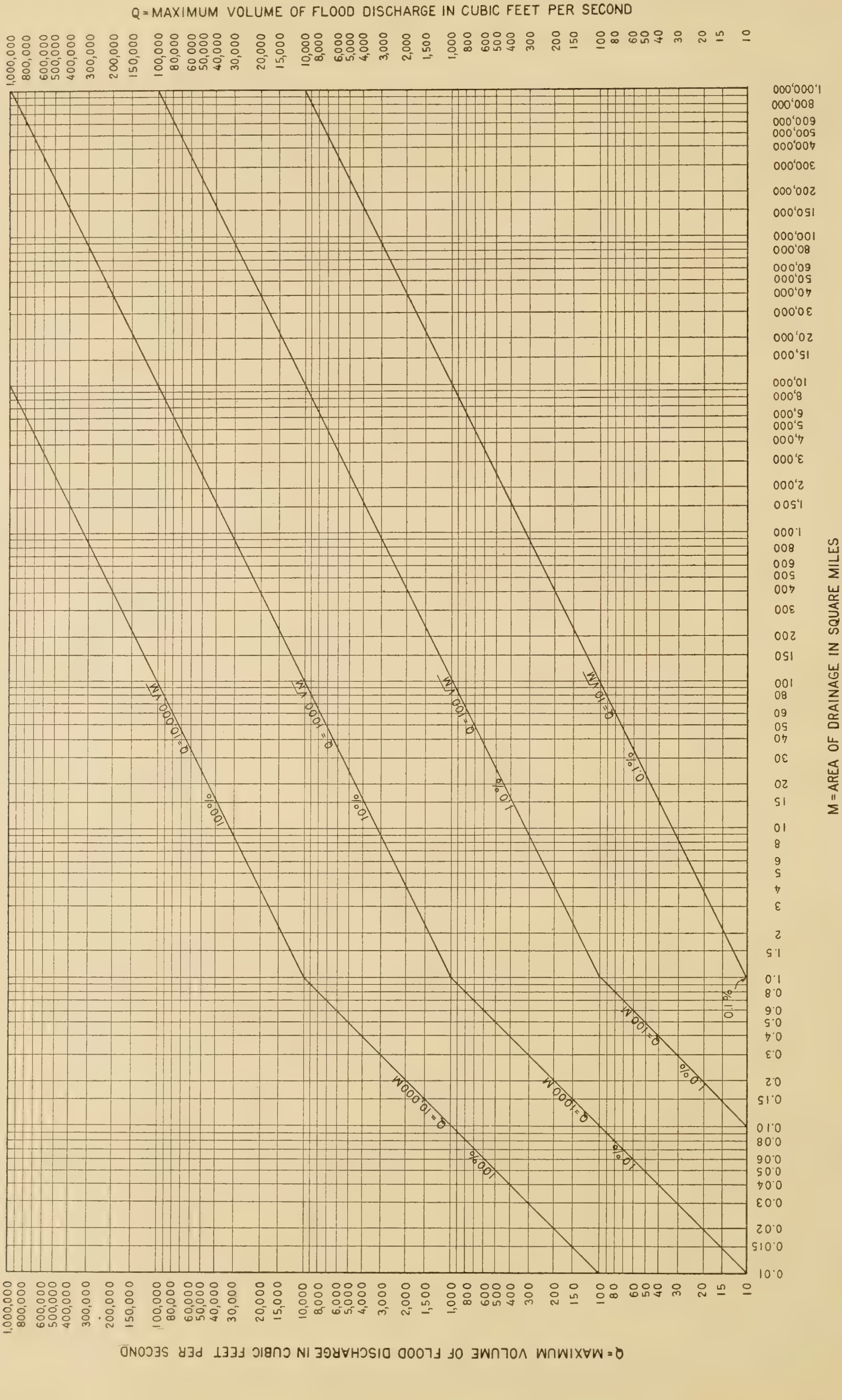

Diagram showing percentage rating on the Myers scale for any given flood discharge

SURVEY OF SOILS AND PAVEMENT CONDITIONS IN PROGRESS IN MICHIGAN

By V. R. BURTON, Engineer on Special Assignments, Michigan State Highway Department

A COMBINED pavement condition and soil survey was begun by the Michigan State Highway Department late in the fall of 1925. The work was discontinued because of extremely wet weather and subsequent freezing. At the time the operations were temporarily halted 108 miles of pavement-condition surveys had been made and 16 miles of general soil surveys were finished. The plans for 1926 include the survey of the remaining 2,000 miles on the State highway system. Whether the work will be accomplished depends to some extent upon the funds available. However, it is the intention to cover during the coming season at least a large variety of soils and pavement designs. Sufficient work has been done already to determine upon adequate methods for collecting and classifying the information; and the 1926 work will be begun with a clear understanding of the organization and methods necessary to complete the survey. Acknowledgment should be made of the material assistance secured from a study of the methods of the Pennsylvania State Highway Department pavement-condition survey.

The building of concrete pavements in Michigan has been going on at an increasing rate for the past 15 years, and it is probable that they will continue to be the predominating type of hard-surfaced road in the State. The designs of pavement slabs vary from the old 6–8–6 cross section with expansion joints at 30-foot intervals, through the 7 and 8 inch uniform thickness with or without expansion joints, to the existing 1924 sections with 10–8–10, 9–7–9, and 9–6–9 dimensions varied to suit the demands of traffic and the character of the subgrade. Various methods of subgrade treatment of the porous subbase type have been used in a more or less experimental way since 1921. Different designs of subdrainage and steel reinforcement have been in use throughout the same period.

The service records of the different types of design throughout the State should furnish some valuable information. It only remains to collect and classify these data for the purpose of deciding upon the relative effectiveness of the various methods of design. Information concerning the methods of construction, materials, and workmanship has been recorded by the State highway department especially on the more recent State-supervised jobs. The pavement survey was projected, therefore, to cover the major portion, if not all, of the pavements laid to date.

Lack of knowledge of the kinds and properties of subgrade soils is the most serious obstacle to a better understanding of the differences in the behavior of pavements laid under apparently identical conditions. It has been felt that this factor must be taken into consideration to make the pavement-condition survey of any great value. It seems to be generally believed that most sandy soils provide a good subgrade while clay soils as a rule are poor subgrade material. The intermediate type, however, with which we are more often concerned seems to be little understood or, worse still, not even recognized.

The decision has been made, therefore, to carry on a soil survey concurrently with the pavement survey for the purpose of determining the proper subgrade and pavement design suitable for a particular soil type under given climatic and traffic conditions. A discussion of this phase of the matter with the official in charge of the land economic survey being carried on in Michigan brought out the fact that about one-third of the State had been covered by some sort of soil survey by that agency and the State agricultural college, both in cooperation with the Bureau of Soils of the United States Department of Agriculture. The older soil maps are, for our purpose, of limited value only since the science of soil mapping and testing has been undergoing almost as rapid a change as that of highway engineering. In addition, the condition and properties of a subgrade soil must be determined in greater detail and from a different viewpoint than even the most recent agricultural soil maps will permit.

BUREAU OF SOILS SURVEYS REDUCE LABOR OF SUBGRADE STUDY

By adopting the Bureau of Soils classification of series and types and profiles it seems possible to diminish to a considerable extent the amount of laboratory testing. For instance the Fox sand is distributed quite generally over this State and differs only slightly throughout the whole area. The recognition of the fact that a given soil type in any particular climate and under similar conditions of drainage is bound to be much the same regardless of the location where it is found is a new idea to the highway engineer and not a very old one to the agricultural soil expert. The work of the Bureau of Soils together with our own State agencies has made soil mapping for highway purposes a practical possibility although this work was conceived and carried out almost exclusively for the uses of agriculture and forestry.[1]

About 100 different types of soil have been identified and mapped in Michigan up to the present time. While this may seem a large number it should be borne in mind that once a type is recognized and tested, very little future testing will be necessary. If every different soil encountered on a highway had to be tested in the laboratory the expense and labor involved would be prohibitive. The value of the Bureau of Soils maps is that they indicate the location of the various types, so that when the subgrade characteristics of all types have been determined the simple identification of the type will, in most instances, be sufficient to stamp the subgrade as good or bad.

The pavement condition survey party consists of one engineer familiar with construction and three assistants. Two of these assistants chain the pavement, marking it at every 20 feet and painting the station number every 100 feet. The third assistant

(Continued on page 92)

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[1] Methods of making soil surveys for highways using Bureau of Soils maps are discussed by A. C. Rose in PUBLIC ROADS, "Practical field tests for subgrade soils," v. 5, No. 6, August, 1924, "Field methods used in subgrade surveys," v. 6, No. 5, July, 1925, and "The present status of subgrade studies," v. 6 No. 7, September, 1925.

TESTS OF CONCRETE IN TENSION[1]

By **A. N. JOHNSON**, Dean, College of Engineering, University of Maryland

THE TESTS here described are the result of cooperative work between the University of Maryland, the State Roads Commission of Maryland, and the United States Bureau of Public Roads.

The purpose was to obtain more data as to the relative strength of concrete specimens in compression and tension, the tension specimens to have a cross-sectional area equal to those of the compression specimens.

To do this required special apparatus which was designed and made in the shops of the University of Maryland. Each tension specimen consists essentially of a cylindrical section, on each end of which inverted frustums of cones were made, so that the total length of the specimen was nearly 21 inches. The cylindrical portion of the specimen was the size of the cylinders used in the compression tests, 9 inches long by 4½ inches in diameter. The tension test specimens were cast in brass moulds which were split their entire length and bolted together. Before casting a specimen, the moulds were rubbed with an oily rag. The concrete was put in from one end and tamped with a rod. As the concrete was placed in the moulds, they were hit with a wooden mallet which resulted in a concrete free from small air pockets. The compression specimens were cast in steel moulds which were made by splitting seamless tubing along one element of the cylinders and holding the edges together by means of bolts attached to lugs. Compression specimens were tamped as the concrete was poured in one end and these moulds also were struck with a wooden mallet.

From a given batch of concrete, three specimens were made to be tested in compression and three in tension. No particular care was taken, however, that subsequent batches should have the same water ratio. The quartz sand used contained 4 to 5 per cent of silt, wet measure, and a trace of inorganic impurities, and had a fineness modulus of 3.28. The coarse aggregate was limestone not exceeding 1 inch in size, with a fineness modulus of 6.20.

APPARATUS FOR TENSION TEST

In the design of the apparatus to hold the tension specimen, special care was taken to insure that the geometric axis of the tension specimen should coincide with the line of force applied to the specimens by the testing machine. For this purpose there had been made a split aluminum cylinder of inside diameter and length equal to the diameter and length of the cylindrical portion of the tension specimen; and two conical, split, brass ends the smaller diameter of which was sufficient to permit them to be passed over the ends of the test specimen. In preparing the specimen for testing the aluminum cylinder was placed around the cylindrical portion and held in place by a wooden clamp. The brass ends, their two halves bolted together, were then passed over the conical ends of the specimen until they rested on flanges provided at the ends of the aluminum cylinder, using this cylinder to center them with reference to the axis of the specimen. There was thus left an annular space between the brass end pieces and the specimen, which was filled with molten rosin poured through holes in the top of the brass ends. As soon as both end pieces had been thus placed, the aluminum centering cylinder surrounding the center portion of the specimen was removed. Reference to Figure 1 will show the arrangement of the specimen and the apparatus. The diagram shows clearly how, by means of links and pins with rocker

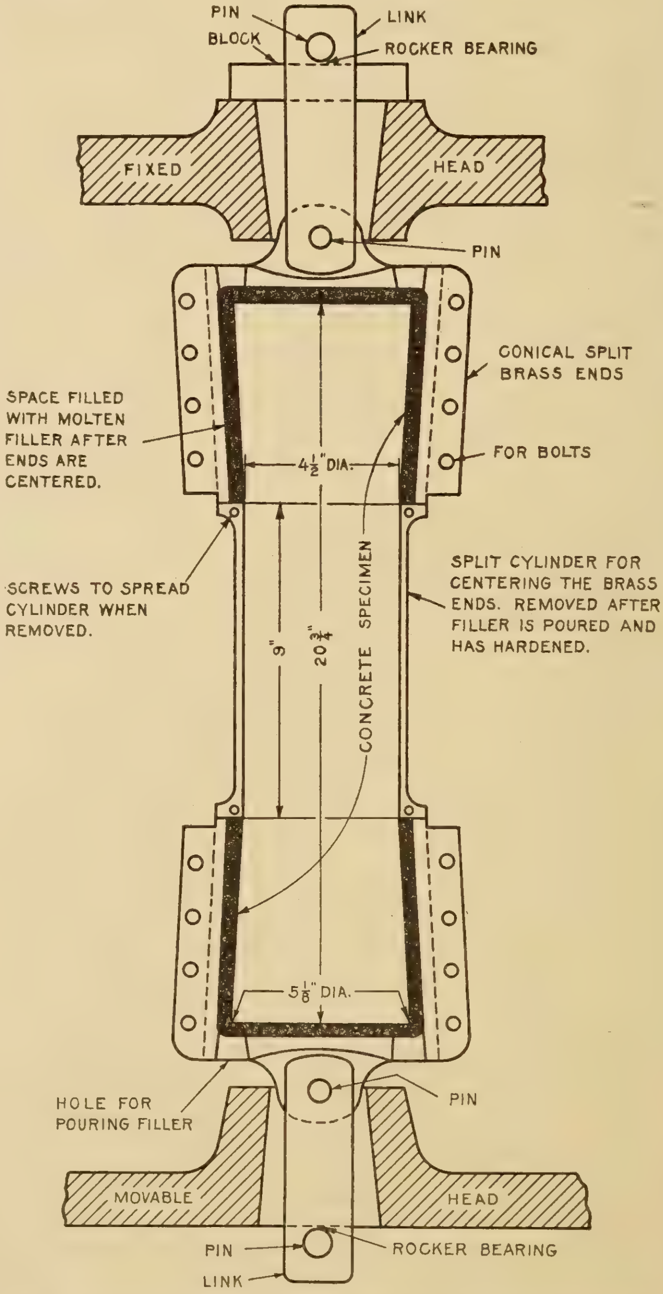

FIG. 1.—Diagram showing the arrangement of the specimen and the apparatus for the tension test

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[1] This paper is substantially the same as a paper presented by the writer at the annual meeting of the American Society for Testing Materials, Atlantic City, N. J., June 22-25, 1926.

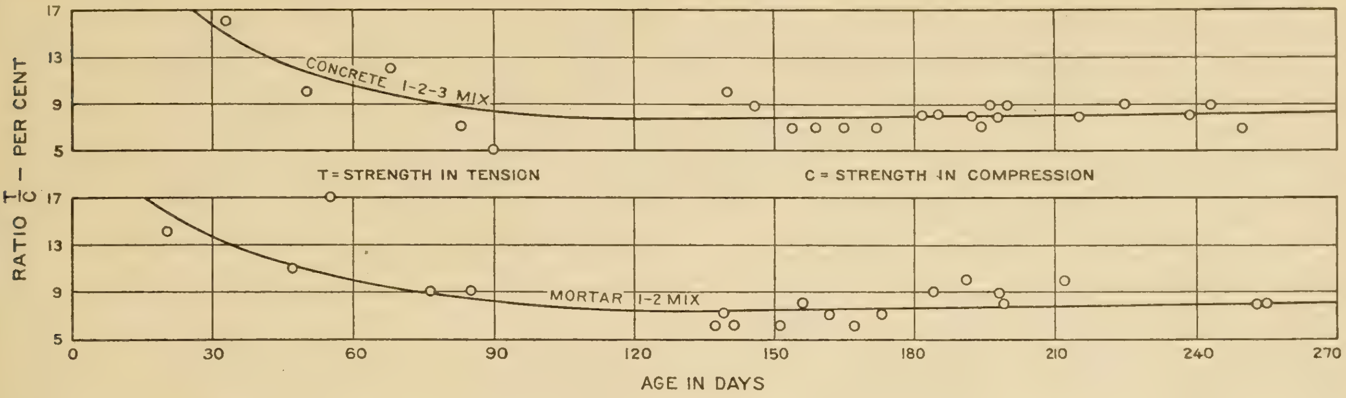

FIG. 2.—Diagram showing the ratio of tensile to compressive strength of concrete and mortar specimens of various ages

bearings, the specimen was mounted in the machine in such a manner that there was no twist or eccentric loading of the specimen during the application of the load.

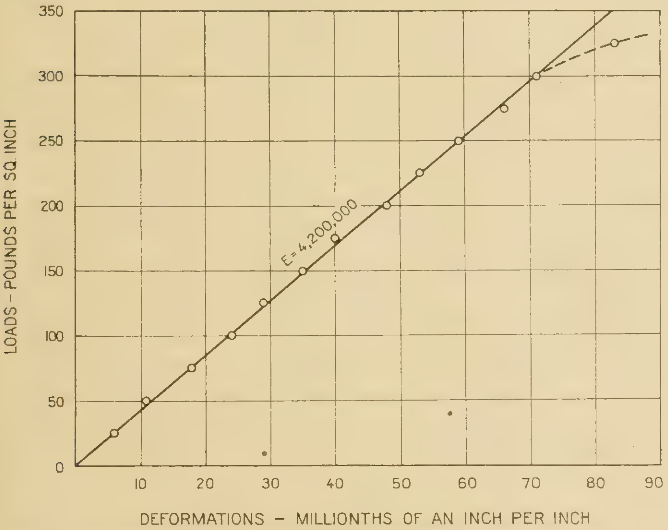

FIG. 3.—Relation of load to deformation of a 1:2 mortar specimen tested in tension

The compression tests were made with a spherical bearing block and usually a very thin cap of plaster was applied to the ends to insure flat bearings.

A 100,000 pound Riehle machine was used for these tests, the load being applied slowly by hand at a rate not exceeding 10,000 pounds per minute. In operation, the apparatus worked very satisfactorily.

The specimens were stored in damp sand for about two weeks and then removed to a room in the laboratory. They were tested at various ages up to 8 or 9 months.

The lack of any attempt to control the water ratio and the comparatively short period of curing accounts for the irregularity in the results when compared with the age of the specimens. As it was the intent primarily to study the ratio of tension to compression strengths, it was not necessary, in fact it was probably better, that there should be a variety in the specimens that would correspond with variations that might be expected in most commercial work.

Practically all the concrete tension specimens broke within or near the middle third of the cylindrical portion of the specimen, whereas a greater number of the mortar tension specimens broke near one clip or the other. In all instances, the breaks were nearly planes perpendicular to the axis of the specimen.

RESULTS OF THE TESTS

The results obtained are detailed in Tables 1 and 2. Table 1 shows the results for the mortar specimens, which were mixed 1 part cement to 2 parts sand. Table 2 shows the results obtained with concrete mixed 1 part cement, 2 parts sand, and 3 parts coarse aggregate. From the tables it will be noted, first, that the ratio of tensile to compressive strength is practically the same for the mortar specimens as for the concrete, and, second, that for specimens up to 90 days old the ratio in each case varies from about 15 per cent to about 8 per cent, and after this period, the ratio becomes practically constant at 8 per cent. It is apparent that during the first 60 to 90 days the increase in tensile strength is not as great as the increase in compressive strength, but beyond this time it appears that both are modified in practically the same ratio. The relation of these results is shown in Figure 2.

The agreement of the results seems so persistent, over such a wide variation in the character of the concrete that there is considerable definite evidence as to the ratio between the tensile strength and compression strength. Thus, in any given case the compression strength being determined, the tensile strength is made known with sufficient accuracy for most purposes.

During these experiments, a few measurements were made to determine the elastic curve of the concrete in tension. It is, perhaps, sufficient here to state a few of the results obtained.

TABLE. 1.—*Results of compression and tensile strength tests of cement mortar, 1:2 mix*

[Cross-sectional area of specimens, 16 square inches (approximately). Each result the average of tests of 3 specimens]

Age	Compressive strength	Tensile strength	Ratio tensile to compressive strength	Age	Compressive strength	Tensile strength	Ratio tensile to compressive strength
	Lbs. per sq. in.	*Lbs. per sq. in.*	*P. c.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*	*P. c.*
20 days	2, 510	349	14	162 days	6, 123	399	7
47 days	3, 827	421	11	167 days	6, 750	431	6
55 days	2, 640	442	17	173 days	7, 133	466	7
76 days	4, 990	434	9	184 days	5, 790	525	9
85 days	4, 200	370	9	191 days	6, 340	630	10
137 days	6, 230	347	6	198 days	5, 420	494	9
139 days	7, 140	489	7	199 days	5, 873	496	8
141 days	7, 183	434	6	212 days	5, 613	541	10
151 days	7, 577	488	6	253 days	6, 663	547	8
156 days	7, 223	583	8	255 days	6, 183	505	8

TABLE 2.—*Results of compression and tensile strength tests of cement concrete, 1:2:3 mix*

[Cross-sectional area of specimens, 16 square inches (approximately). Each result the average of tests of 3 specimens]

Age	Compressive strength	Tensile strength	Ratio tensile to compressive strength	Age	Compressive strength	Tensile strength	Ratio tensile to compressive strength
	Lbs. per sq. in.	*Lbs. per sq. in.*	*P. c.*		*Lbs. per sq. in.*	*Lbs. per sq. in.*	*P. c.*
18 days	1,297	--------	--------	181 days	1,847	155	8
33 days	2,160	349	16	185 days	4,280	326	8
50 days	3,197	335	10	192 days	3,773	301	8
69 days	3,043	360	12	194 days	4,667	307	7
83 days	3,007	220	7	196 days	2,130	192	9
90 days	2,503	126	5	197 days	2,863	235	8
140 days	3,353	334	10	200 days	2,503	236	9
146 days	4,160	369	9	215 days	4,260	330	8
154 days	4,175	293	7	225 days	3,337	298	9
159 days	3,310	244	7	239 days	3,647	284	8
165 days	3,473	257	7	243 days	2,797	247	9
172 days	3,023	221	7	249 days	3,373	248	7

The deformations in tension were observed by means of a mirror extensometer attached in a manner similar to that described for observations of compression cylinders in a paper by the author to be found in the A. S. T. M. Proceedings, volume 24, 1924, part 2, page 1025.

As illustrative of some of the results obtained, Figure 3 shows the deformations obtained upon a 1:2 mortar specimen, which broke at 340 pounds per square inch, and indicates a modulus of elasticity of 4,200,000 pounds per square inch. The values of E for the few specimens tested varied from 3,300,000 to over 5,000,000 for some specimens.

It is the expectation to carry on a more extended series of observations of the modulus of elasticity of concrete in tension, particularly with Lumnite cement.

(Continued from page 89)

accompanies the engineer following the chaining party. The engineer and this assistant walk along opposite sides of the pavement locating the cracks and sketching them on specially prepared cross-section paper. All surface defects, such as pitting, scaling, spalling, hair checks, etc., are noted on the sketch. About 4 miles of pavement is an average day's work for this party.

A soil surveyor skilled in soil mapping follows the pavement party and uses the painted stations to locate the boundaries of the various soil types. A strip of soil 200 feet wide is mapped. After the mapping is completed the soil surveyor returns over the same route and selects typical soil samples which are sent to the laboratory for routine tests. A sufficient number of samples are tested to determine the extreme upper and lower limits of variation within the type.

Both the pavement condition and the soil surveys are forwarded to the main office immediately upon completion of the field work and the information is transferred to the original plan which is on a scale of 100 feet to the inch. This sketch is easily readable when the work is done by a neat draftsman and the complete results may then be easily studied or compared. The type of soil, and depth of the roadway in the soil profile, the grades, cross section, surface and subdrainage are all noted and may be read at a glance. Surface data sheets of the pavement survey are prepared in such a way that separate sections of road may be compared slab by slab. The surface data sheets give the slab lengths, number and kind of cracks, the pavement classification based upon the length of the unbroken slab and the area of the pieces of broken slab, the amount of scaling, spalling, and hair checking, etc.

A study of one phase of the soil survey—the subsidence of fills in peat marshes—has been practically completed. The fills were cross-sectioned by borings so as to determine the actual shape of the floating fill. The classification of the peat strata was made in accordance with the system developed largely by Dr. Alfred P. Dachnowski, of the Bureau of Plant Industry of the United States Department of Agriculture. Testing of peats in accordance with the standard soil methods is impossible since oxidation takes place unless the sample is kept continually moist with the result that a totally different class of material remains. No screen analysis is practicable and shrinkage tests and bearing values must be obtained from the moist material without initial compression.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

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

ANNUAL REPORT

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

DEPARTMENT BULLETINS

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

DEPARTMENT CIRCULARS

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

MISCELLANEOUS CIRCULARS

No. 60. Federal Legislation Providing for Federal Aid in Highway Construction.
62. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-aid Highway Projects.

FARMERS' BULLETINS

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

SEPARATE REPRINTS FROM THE YEARBOOK

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

OFFICE OF PUBLIC ROADS BULLETIN

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

OFFICE OF THE SECRETARY CIRCULARS

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

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

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

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

MAY 31, 1926

GOVERNMENT PRINTING OFFICE

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2,863,197.86	611.8	$ 11,644,803.06	$ 5,596,523.62	665.6	$ 5,469,336.08	$ 2,579,949.31	231.1				$ 3,309,784.21	Alabama
Arizona	9,580,133.43	5,016,119.94	613.8	1,230,100.53	748,233.76	102.3	1,537,643.65	1,043,510.75	95.0	$ 82,189.48	$ 50,225.99	12.2	2,759,158.56	Arizona
Arkansas	13,310,190.08	5,380,181.73	1,048.9	4,643,529.91	2,081,104.37	261.5	4,940,580.79	2,370,447.28	306.8	536,670.32	236,160.00	37.4	1,537,910.62	Arkansas
California	22,346,175.99	10,719,249.61	894.8	4,513,257.39	2,114,444.58	150.9	11,170,314.24	5,518,926.90	312.8	604,506.02	245,734.28	7.6	3,474,459.63	California
Colorado	11,876,703.94	6,067,814.34	651.2	1,278,321.21	665,157.96	61.8	4,929,735.32	2,463,212.49	245.5	423,072.26	188,359.34	13.1	2,941,267.87	Colorado
Connecticut	4,558,639.29	1,819,368.66	101.6	855,927.90	281,217.14	15.5	2,359,159.77	718,923.03	34.6	1,592,001.75	371,404.12	21.6	1,142,768.05	Connecticut
Delaware	4,281,559.81	1,496,190.65	107.1	636,492.48	285,474.95	17.2	1,115,536.81	470,577.90	28.2	496,458.35	204,710.90	12.2	17,103.60	Delaware
Florida	2,959,273.72	1,405,487.97	96.3	723,609.59	344,533.96	36.5	9,725,838.00	4,613,485.95	272.7				1,721,446.12	Florida
Georgia	20,156,002.37	9,406,366.46	1,478.3	4,576,128.77	2,232,012.73	302.6	11,833,250.23	5,836,816.37	664.7	1,781,810.07	850,105.71	31.7	106,651.73	Georgia
Idaho	9,394,676.80	4,815,332.26	600.1	1,525,282.72	954,548.38	115.8	2,622,706.72	1,643,810.45	132.2	1,134,789.56	648,430.81	77.2	497,505.10	Idaho
Illinois	40,010,481.10	18,640,076.28	1,236.2	4,065,949.17	1,961,514.46	140.3	5,311,686.37	2,552,427.37	177.3				6,678,179.89	Illinois
Indiana	13,639,172.65	6,562,455.68	422.1	3,023,955.71	1,467,991.44	102.6	16,255,226.39	7,587,045.09	437.7	448,910.56	222,014.00	10.7	2,364,848.79	Indiana
Iowa	27,272,285.21	11,107,492.99	1,996.9	1,640,794.87	755,809.11	113.9	8,655,167.73	4,218,730.07	567.7	2,979,460.29	1,416,228.69	198.6	1,987,302.14	Iowa
Kansas	26,399,695.77	9,755,273.32	831.4	6,426,905.87	2,835,215.93	329.1	11,014,045.22	4,371,309.59	602.3	1,688,438.64	557,846.70	113.6	1,944,765.46	Kansas
Kentucky	14,832,324.28	6,205,994.59	584.9	4,667,594.35	1,756,029.10	123.7	6,516,469.46	3,063,970.19	320.6	299,364.07	149,682.03	4.6	2,037,133.09	Kentucky
Louisiana	11,939,424.97	5,279,870.86	927.6	1,891,167.71	867,535.71	128.3	3,311,151.85	1,605,168.11	149.4				1,519,833.32	Louisiana
Maine	8,174,281.31	3,907,870.33	281.4	573,271.45	284,637.06	22.2	1,809,495.54	706,814.24	57.5	1,526,777.08	525,467.78	39.3	1,040,038.59	Maine
Maryland	8,132,506.90	3,849,383.15	294.4	2,792,436.20	1,263,608.07	128.9	169,995.00	84,997.50	10.8	1,561,681.01	716,133.95	74.7	10,934.33	Maryland
Massachusetts	14,047,656.22	5,467,661.28	300.6	3,985,743.02	1,110,609.34	68.6	3,550,349.48	963,029.43	46.1	493,449.82	140,235.00	7.6	2,427,190.95	Massachusetts
Michigan	16,234,000.80	7,328,316.91	612.6	9,763,239.98	4,498,735.39	350.4	9,263,608.64	4,182,702.41	217.7	105,818.00	47,736.00	3.1	4,284,874.29	Michigan
Minnesota	30,415,685.89	12,738,642.04	2,721.2	5,271,152.99	2,284,974.52	397.0	9,821,971.75	2,975,500.00	629.6	464,657.41	34,000.00	15.0	1,558,663.44	Minnesota
Mississippi	10,292,285.79	4,988,702.73	803.4	3,910,793.14	1,954,397.32	278.9	7,120,916.68	3,529,463.68	369.2	2,168,639.90	992,466.87	111.6	662,987.40	Mississippi
Missouri	17,368,156.57	8,219,411.43	1,118.9	10,661,050.09	5,094,711.47	395.9	20,377,381.96	8,054,229.54	557.3	2,256,076.18	949,782.63	57.8	468,300.93	Missouri
Montana	10,156,600.41	5,317,523.15	921.6	1,244,383.40	1,015,942.74	133.3	1,523,431.87	1,008,165.06	120.4	836,221.14	456,127.69	97.7	5,627,126.36	Montana
Nebraska	9,306,374.36	4,339,523.50	1,570.6	2,169,810.65	1,056,279.01	195.0	12,109,329.60	5,998,839.67	1,215.4	1,541,463.70	769,987.45	164.0	2,421,205.37	Nebraska
Nevada	4,917,465.69	3,088,299.78	357.3	2,640,729.82	2,042,634.81	181.5	3,136,209.75	2,698,004.08	328.5				966,276.33	Nevada
New Hampshire	4,165,687.86	1,986,226.87	208.1	826,870.74	391,223.20	29.5	582,611.55	274,125.52	17.7	241,941.63	117,707.65	8.2	400,208.76	New Hampshire
New Jersey	11,961,357.45	3,820,679.99	219.1	4,227,678.25	1,236,143.72	68.5	6,746,521.08	2,515,017.44	31.3	240,608.13	45,765.00	3.1	849,813.85	New Jersey
New Mexico	8,717,999.18	4,914,070.61	1,081.3	3,568,872.54	2,326,079.86	329.6	1,456,200.02	966,613.62	116.3	112,357.05	70,735.50	5.6	2,694,886.41	New Mexico
New York	28,597,769.67	12,229,076.53	831.5	12,579,727.57	4,913,674.20	314.7	32,079,571.58	9,123,202.24	577.7	7,890,953.00	1,765,990.00	114.3	6,013,252.03	New York
North Carolina	21,014,450.41	8,746,454.59	1,119.8	5,994,969.06	2,430,883.35	138.1	7,387,341.40	3,160,668.31	165.9	1,283,847.15	641,923.57	50.4	737,276.18	North Carolina
North Dakota	10,829,263.82	5,208,930.47	1,917.5	1,455,792.12	748,801.58	275.6	4,382,686.58	2,236,028.14	570.9	3,840,966.05	1,876,966.93	493.3	617,931.86	North Dakota
Ohio	41,572,252.81	15,244,993.93	1,191.1	6,059,391.44	2,107,214.63	171.8	9,432,894.33	3,653,206.34	293.8	3,927,570.42	1,381,899.48	103.2	3,344,481.62	Ohio
Oklahoma	20,787,024.94	9,672,890.34	852.2	6,496,136.84	3,025,537.08	278.7	3,949,958.99	1,877,399.11	140.8	482,026.22	235,088.31	52.3	1,248,872.16	Oklahoma
Oregon	14,388,188.70	7,142,364.63	794.6	2,325,783.62	1,256,645.21	111.3	3,462,508.87	1,913,780.24	159.5	519,886.14	303,949.91	22.2	262,607.01	Oregon
Pennsylvania	43,054,835.19	16,222,023.97	850.3	17,824,416.03	5,169,421.37	325.5	25,155,981.30	7,060,697.63	512.9	3,107,544.09	1,032,120.89	67.5	1,854,517.14	Pennsylvania
Rhode Island	2,628,496.20	1,119,688.09	64.8	1,360,119.89	439,140.97	21.9	1,531,802.80	427,155.00	28.5	45,705.55	13,050.00	0.9	668,534.94	Rhode Island
South Carolina	11,163,347.84	5,121,267.54	1,236.9	3,560,390.58	1,524,936.87	202.3	6,288,755.22	2,806,298.87	272.3	339,304.66	143,936.58	23.5	205,084.14	South Carolina
South Dakota	12,091,434.67	5,989,879.00	1,447.9	4,801,631.44	2,329,698.81	657.5	3,582,932.31	1,722,393.53	570.4	482,685.86	262,655.48	104.2	862,163.18	South Dakota
Tennessee	13,789,140.98	6,732,079.77	497.9	7,584,709.49	3,419,113.70	265.4	7,900,200.62	3,625,744.50	265.1	616,648.03	259,926.86		1,243,726.17	Tennessee
Texas	54,120,970.83	21,057,940.12	3,907.1	14,578,815.45	6,146,595.22	942.7	18,773,855.28	8,311,683.32	961.1	3,243,754.71	1,459,817.79	86.0	3,630,394.55	Texas
Utah	6,259,159.41	3,818,836.91	423.1	1,543,632.88	979,511.08	90.0	2,149,723.47	1,573,821.69	187.5	385,929.62	297,447.91	27.4	1,149,161.41	Utah
Vermont	3,015,174.51	1,452,894.45	107.8	1,226,868.13	564,805.06	26.7	1,384,349.78	526,371.94	29.2	401,743.24	171,376.19	5.2	553,059.36	Vermont
Virginia	13,099,720.01	6,271,998.20	676.2	8,729,465.99	4,033,198.19	323.3	6,673,203.06	2,925,727.08	193.1	458,694.60	217,711.77	14.0	52,878.76	Virginia
Washington	13,352,504.18	6,117,211.87	526.7	3,699,085.54	1,654,697.59	141.3	3,233,193.60	1,615,600.00	32.1	345,788.76	129,000.00	16.4	629,266.54	Washington
West Virginia	7,343,200.86	3,230,293.33	326.7	1,639,091.90	673,572.94	57.0	5,659,295.82	2,250,710.76	143.6	1,049,669.28	450,047.64	39.6	747,886.33	West Virginia
Wisconsin	21,807,140.91	8,910,640.62	1,451.7	2,994,612.24	1,446,735.85	137.4	5,633,325.43	2,774,909.19	251.5	1,734,700.38	836,574.00	94.5	3,458,955.34	Wisconsin
Wyoming	8,809,819.33	4,739,096.67	982.0	2,087,000.03	1,282,218.38	151.5	2,553,262.00	1,610,353.02	193.7	149,528.34	95,896.00	20.5	838,610.93	Wyoming
Hawaii							1,050,897.83	312,635.18	15.9				787,517.82	Hawaii
TOTALS	740,140,790.82	325,654,346.00	41,898.3	211,521,493.75	93,653,725.79	9,869.6	336,701,611.92	144,123,599.13	13,860.9	53,924,308.52	21,584,526.42	2,463.6	86,358,802.66	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 77,031,615.32 Federal aid $ 33,660,927.65 Miles 3,018.2