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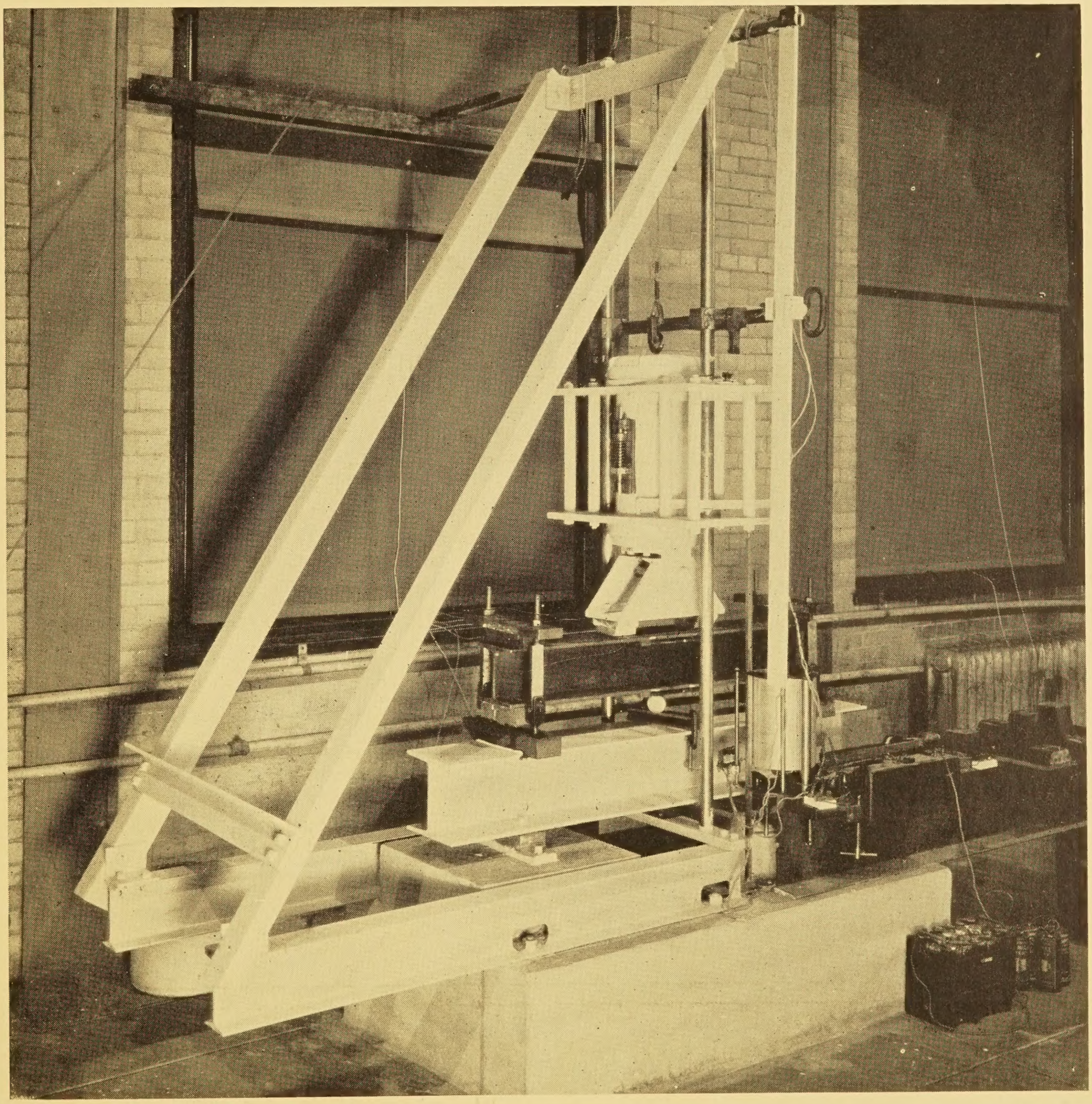

Impact Machine Used in Tests of Concrete Beams

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STATIC AND IMPACT STRAINS IN CONCRETE

A COMPARISON OF THE MODULI OF RUPTURE OF BEAMS UNDER STATIC AND IMPACT LOADS

Reported by J. T. THOMPSON, Highway Research Specialist, Bureau of Public Roads [1]

IN THE summer of 1923, the Bureau of Public Roads brought to completion a series of impact tests on a large number of concrete slabs. These tests, which were conducted at the Arlington Experiment Farm, were subsequently reported in the issue of PUBLIC ROADS for April, 1924.[2] Among the striking features of this report were the exceptionally high unit fiber tensile deformations recorded. The average deformation for slabs of 1:1½:3 concrete was 0.000270 inch per inch; and since the average modulus of elasticity of the concrete was 4,480,000 pounds per square inch the indicated unit fiber stress in tension at elastic failure was 1,210 pounds per square inch. This modulus of rupture was recognized as being at least twice as great as the static modulus for the same concrete.

Apparently the discrepancy thus indicated was capable of but two explanations; either the strain gages used for measuring the unit deformations under impact were inaccurate, or the concrete subjected to impact was actually strained to a greater degree without rupture than is possible under static loads.

In order to throw some light, if possible, upon this question the Bureau of Public Roads and the Johns Hopkins University entered into a cooperative research project the aims of which were to study, in general, the action of plain concrete beams under impact forces and specifically to compare the static and impact moduli of rupture.

The economic importance of answering this question will be apparent when one considers that recent researches have evaluated the impact force delivered by motor vehicles to pavement slabs and shown it to be certainly double, and under some conditions several times the static weight of the vehicle. The natural tendency, in view of this information, is to increase pavement thickness; but if it is true that there coexists with the increase in force due to dynamic action a compensating possibility of stressing the pavement further without rupturing it, the increase in thickness might be unnecessary.

TESTS SHOW DURATION OF LOAD TO BE THE CRITICAL FACTOR

The tests prove conclusively that the graphic strain gauges used are free from inertia effects within the range of the use to which they were put; and the indications are that the deformations measured with similar instruments in the slab impact tests of the bureau are substantially correct as recorded by the instruments.

Doubt of the accuracy of the instrument being thus set at rest the experiment shows further that plain concrete beams subjected to static load have a considerably lower modulus of rupture than exactly similar beams subjected to successive impacts of increasing magnitude, caused by dropping a weight upon the beam from progressively increasing heights, when the weight is dropped but once from each height. Under these conditions, which are similar to those prevailing in the slab impact tests of the bureau, in which the weight was dropped only five times from each height, the beams resisted without failure impact forces considerably in excess of the static loads which caused failure. In other words, they were capable of greater elastic deformation under impacts applied as described than under ordinary static load applications.

When, on the other hand, the beams were subjected to repeated applications of impact of the same magnitude, the greatest impact they would successfully withstand was that which produced a tensile stress equal to about 55 per cent of the static modulus of rupture, indicating that the behavior of the concrete under impact is very similar to its behavior under static loads, as evidenced by static fatigue tests.

Below the fatigue limit it has been shown that equivalent loads, without regard to the manner of their application, whether static or impact, produce the same stress; and the indications are that the cause of the different behavior beyond that limit is to be found in the duration of the load application. This is evidenced by the fact that beams subjected to static loads for 10 seconds at a time showed the lowest modulus of rupture and impacts of an estimated duration of 0.015 second the highest with an intermediate value corresponding to impacts of an estimated duration of 0.030 second.

As an incidental result of the tests it has been shown that the variation in stress from the top to the bottom of a plain concrete beam may not be represented by a straight line but that the error involved in that assumption is small for ordinary safe working loads. It is also indicated that the impact force corresponding to a given height of fall is much greater when the fall is cushioned by worn solid rubber tires than when the cushioning medium is a new tire, thus accentuating the need for smooth pavement surfaces.

TESTS LOADS APPLIED IN THREE WAYS

Thirty-six plain concrete beams all of the same mix, with necessary control cylinders, were used in the tests. About one-third of the number were loaded at the third-points of a simply supported span by a 50,000-pound Olsen testing machine; with measurements of the load, center deflection and unit deformations of the concrete. These are referred to as the "static tests," although it should be borne in mind that "static" loads which are at their maximum intensity for a period of several seconds are in reality very slow "impact" loads.

Another group of the beams, one-third of the total number, was tested in impact by simply supporting the beams on suitable rests and allowing a heavy hammer, cushioned with full-thickness, unused, solid rubber tire segments [3] to fall from various heights, applying the the load to the beams at the third-points. The force of the blow, the deflection of the beam and the unit deformations of the concrete were measured.

[1] Also associate professor of civil engineering at the Johns Hopkins University.
[2] "Impact Tests on Concrete Pavement Slabs," Leslie W. Teller, PUBLIC ROADS, vol. 5, No. 2, April, 1924.

[3] The tire segments used in the tests were generously supplied by Morgan & Wright (U. S. Tires), Detroit, Mich.

The remaining third were tested in impact in exactly the same manner as above except that the tire segments had part of the rubber removed to simulate a badly worn condition.

The reason for employing two conditions of impact—namely, the thick and the thin cushions was to introduce the time element—i. e., the duration of the force—the feeling being that the modulus of rupture, if different for static and impact conditions, would be found to be a function of the rapidity of application.

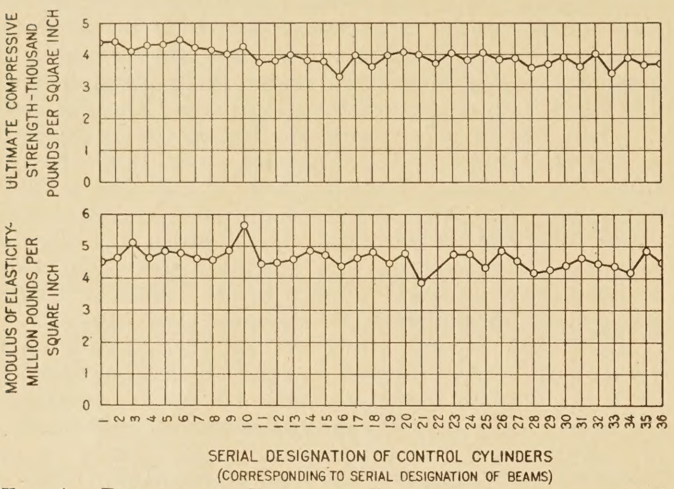

FIG. 1.—DATA FROM TESTS OF CONTROL CYLINDERS. TWO CYLINDERS WERE TESTED FOR EACH BEAM AND THE PLOTTED POINTS REPRESENT THE AVERAGE VALUES FOR THE TWO CYLINDERS

TEST BEAMS AND CONTROL CYLINDERS

The preliminary steps involved the preparation of test beams and control cylinders, the design and construction of an impact testing machine, and the selection and calibration of instruments of measurement. It was thought at the beginning that two beams a week were all that could possibly be tested and therefore only two a week were cast, since it was desirable that they all be of the same age when tested. This meant a lapse of 18 weeks between the casting of the first and thirty-sixth beam in which interim there would be abundant opportunity for differences in proportioning as well as in technique of mixing and placing unless great care were exercised.

Accordingly, enough material was procured for the whole job. The sand and gravel were dried bone-dry and stored, and all the cement was thoroughly mixed to eliminate possible differences between bags. The uniformity of the concrete was governed by controlling the water-cement ratio. In addition to this everything that went into the mixer was weighed and the fineness modulus of the aggregate was carefully checked for uniformity. The technique of mixing, placing, and curing was, as nearly as possible, identical in every case, and Figure 1 shows that a high degree of uniformity in the product was actually obtained.

The thirty-six 6 by 8 by 44 inch plain concrete beams of a mix closely approximating 1 : 1½ : 3 [4] were cast; and for each beam two 6 by 12 inch control cylinders were made, making a total of 72 cylinders in all. The beams were cured under sand in a damp room, and were allowed to dry out at room temperature for one week prior to test. The cylinders were shipped to the Bureau of Public Roads, where they were cured in a similar manner and tested for crushing strength and modulus of elasticity on the test dates of the beams.

THE IMPACT TESTING MACHINE

The essential features sought in the design of the impact machine were: (1) A device that would drop a cushioned weight upon a simply supported beam from a predetermined height; (2) a method of support for the beam that would not restrain it and at the same time would prevent it from moving about when struck; and (3) a means of supporting deflection gauges that would preclude their movement relative to the beam.

The machine (fig. 2), which was built in the Johns Hokpins University shop, consists of a pile-driver-like arrangement equipped with a 265-pound hammer which can be dropped upon the beam from a variable height depending upon the setting of a trigger. The hammer is cushioned by two segments of a 3½-inch solid rubber tire mounted in such a way as to permit them to adjust themselves should there be a tendency for one to apply more load to the beam than the other. These apply the load to the beam at points about 14 inches apart, that is, at about the third points, the idea being to bring about a constant bending moment, and hence constant fiber stress, across the strain gauges located in the recesses shown at the center of the beam.

The test beam is supported at each end upon a cast-iron block, the lower side of which carries a half round running at right angles to the length of the beam. This half round fits into a V notch in a second cast-iron block, which is securely fastened to a heavy steel H beam. This "rocker" support permits the beam to deflect freely, while at the same time the angles shown on each side of the test beam in the center-line cross section (fig. 2) prevent it from moving laterally when struck. No tendency for the beam to move longitudinally was observed. It was at first thought necessary to hold the test beam down against rebound by means of shackles such as appear in the photograph on the cover, but these were found to restrain the beam and were therefore abandoned immediately in favor of the simpler and very satisfactory scheme of using the angles mentioned above.

The support for the deflection gauges is arranged for by inserting the 8-inch H beam between the block supports and the foundation. Since the H beam is supported upon knife rests directly under the reactions applied to it by the supports of the test beam it does not deflect. Only after careful verification of this fact by actual measurement was it used as a rest for the deflection gauges, however.

INSTRUMENTS OF MEASUREMENT

In both the static and impact tests the deformation of the concrete beams was measured by means of 12 graphic strain gauges arranged in slots cast in the concrete, 6 on each side, evenly spaced from top to bottom of the beam. These gauges, developed in the Bureau of Public Roads, are described in detail elsewhere.[5] They consist of a metal bar about 6 inches long and three-eights by one-quarter inch in cross section. At one end is an adjustable fixed point, at the other end a

[4] Acknowledgement is made of the valuable assistance rendered by Malcolm R. Gilpin, senior civil engineering student at the Johns Hopkins University, who volunteered his services in the preparation of these beams and cylinders.

[5] "Pocket Strain Gauge Gives Stresses in Concrete Roads," A. T. Goldbeck, Engineering News-Record, Mar. 29, 1923.

movable point which presses eccentrically against a small shaft. The shaft, thus made to rotate, carries a 4-inch light metal arm at the end of which is a small stylus which records its movement upon a small smoked-glass slide.

In order to settle this question the following method was adopted. A steel H beam was placed in the impact-testing machine and strain gauges were mounted on both tension and compression flanges between steel blocks securely fixed to the beam. The H section was

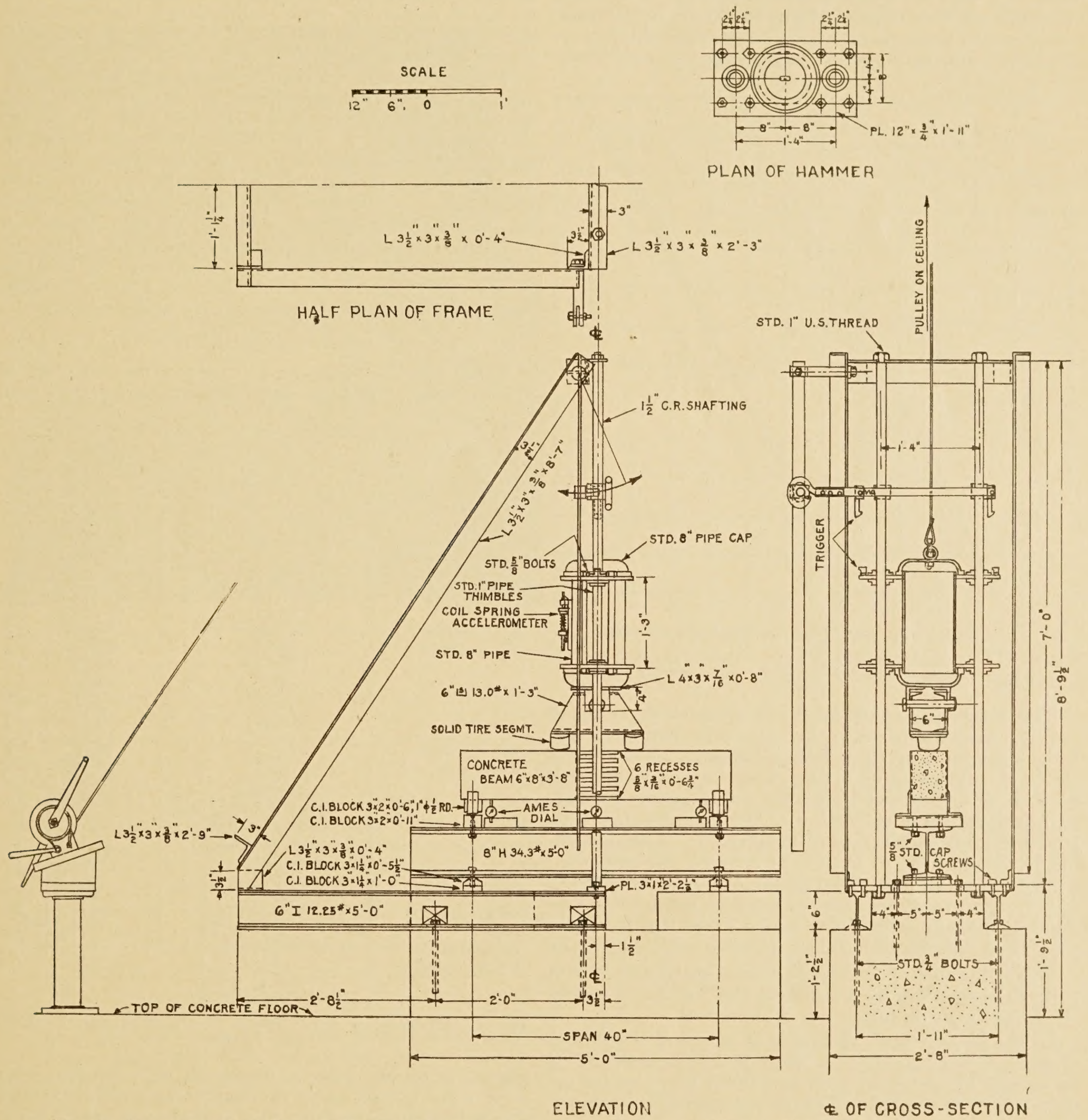

FIG. 2.—DETAILS OF THE IMPACT TESTING MACHINE USED FOR DETERMINING IMPACT STRESSES IN CONCRETE

As previously intimated, it had been suspected since the development of these gauges that they were subject to inertia effects when used in impact—i.e., that they either lagged behind the true movement in tension or continued to record in compression after the impulse had ceased.

so chosen that the product of its modulus of elasticity and its moment of inertia was the same as for the concrete test beams. This was done in order that the records of the strain gauges might be verified under conditions exactly similar to those to be imposed upon them in test.

The H beam was loaded statically by a screw jack and the strain-gauge records plotted against center deflections. The same deflections were then produced by impact and a similar curve was plotted. The curves of Figure 3, typical of several secured, show the results. It will be noted that the impact and static curves check very closely within the limits of deformation imposed upon them, which limits, incidentally, are about the same as those met with under test conditions.

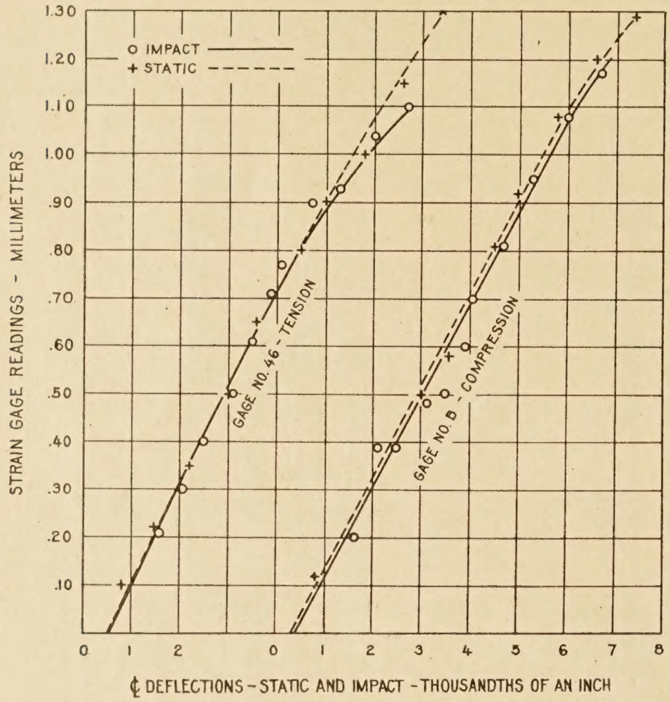

FIG. 3.—COMPARISON OF IMPACT AND STATIC GRAPHIC STRAIN GAGE READINGS SHOWING ABSENCE OF INERTIA EFFECT

This method of checking the instrument depends, of course, upon the premise that identical fiber deformations exist for a given deflection whether produced by static or impact force. This would seem to be true when it is considered that fiber deformation depends upon the radius of curvature of the bent beam. This is a well-known principle of mechanics. Also, assuming the beam bent to the arc of a circle, the measured deflection is merely the middle ordinate of a fixed chord. For every deflection (middle ordinate) there must be one and only one radius. From this it follows that fiber deformation varies directly with deflection.

The deflections of the beams in both static and impact tests were recorded by Ames dials reading direct to ten-thousandths of an inch.

In the impact tests the force of the blow was determined by the product of the mass of the hammer and its maximum deceleration as measured by a coil spring accelerometer the details of which are shown in Figure 4. The general type of this instrument is fairly common. It consists of a weighted plunger mounted vertically in two bearings, so that it is free to move downward against the action of a coil spring. The instrument was rigidly attached to the hammer of the impact machine and when the latter was decelerated upon striking the beam, the plunger of the accelerometer, accelerated downward with respect to the spring forced the latter to compress. The amount of this compression, which is a measure of the deceleration of the hammer, was recorded upon a smoked-glass plate by a stylus attached to the plunger.

Although crude in appearance and perhaps in scientific principle this instrument functioned perfectly for about a quarter of a million blows without once even so much as getting out of adjustment, thereby amply justifying its use in preference to more delicate types which offered themselves.

The calibration of the accelerometer was effected through the use of space-time curves. This method, which has been described elsewhere in detail,[6] consisted of the analysis of a curve, drawn by a stylus attached to the falling hammer, upon a sheet of paper wrapped on a drum rapidly rotating on a vertical shaft (see cover illustration). Concurrently a time curve was drawn on the paper by another stylus whose vibrations were governed by an electrically operated tuning fork of known period. Decelerations determined by analysis of the space-time curve were plotted against accelerometer records and a calibration curve thus obtained.

DESCRIPTION OF THE TESTS

Static tests.—These tests involved nothing out of the ordinary. A beam was supported, as shown in

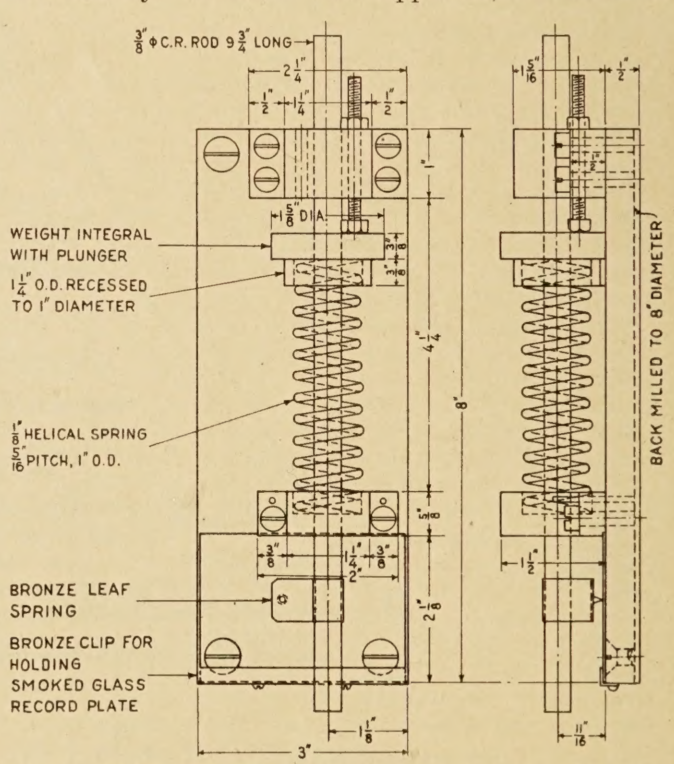

FIG. 4.—DETAILS OF COIL SPRING ACCELEROMETER USED IN THE TESTS

Figure 5, on the same supports as were subsequently used in the impact tests, the load being applied through a spherical head resting on the same tire segments as later were used in impact. The deflection gauge was carried by a yoke suspended from an ordinary "neutral-axis" bar.

The loads were applied at slow speed in increments of about 300 pounds, the load being released after each application at which time the strain gauge slides were

[6] "Accurate Accelerometers Developed by the Bureau of Public Roads," Leslie W. Teller, PUBLIC ROADS, vol. 5, No. 10, December, 1924.

moved up. The quantities recorded were loads, deflections, and unit deformations of the concrete.

Impact tests.—Most of the beams tested under impact were hit but once from each height of drop, the height increasing progressively until the beam failed. The trigger was set so as to bring about the desired

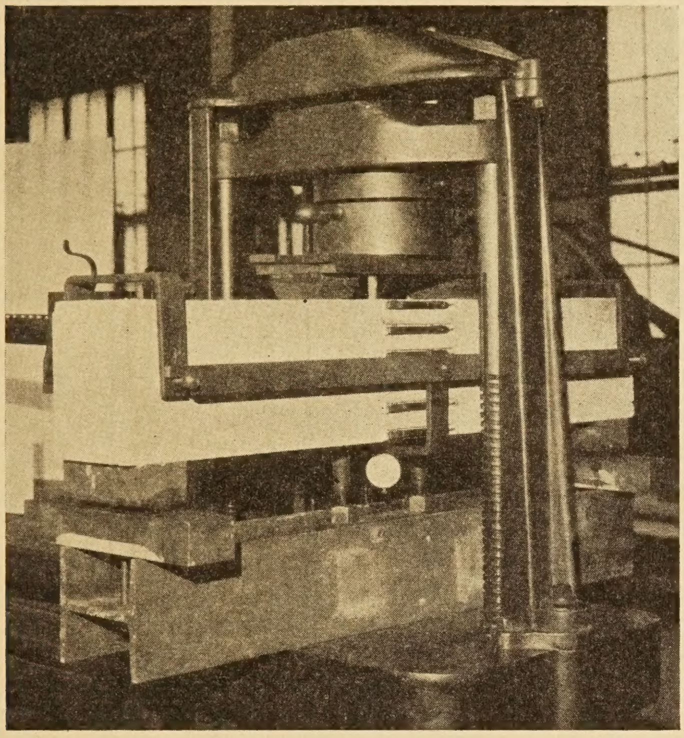

Fig. 5.—Concrete Beam in Test Machine Ready for Static Test Showing Arrangement of Deflection Gage and Graphic Strain Gages

height of drop, the hammer raised by the winch until it was suspended from the trigger hooks, and then, after all gauges had been adjusted, the hammer was suddenly dropped by pulling the releasing lever. The deflections were recorded by "choked," ten-thousandths-type Ames dials, one dial being always used under the center of the beam, and frequently two others being used as near the supports as possible merely to detect any relative movement between the test beam and the H-beam. The strain-gauge slides as well as the smoked-glass plate on which the accelerometer recorded were removed at the end of the test and read.

The effect of what might be termed "impact fatigue" was tried upon a few of the beams. In these tests the technique was similar to that just escribed except that a great many blows were struck from each height of drop. Readings were taken only when thought necessary, sometimes after a few blows, sometimes at the end of a series of several thousand.

The principal thing sought in each case was the modulus of rupture. In order to establish it, the unit deformations, measured at intervals above and below the center of the beam, were plotted to show the variation of strain. The fiber strains were then taken from these curves and plotted against load.

Before trying to ascertain from the strain-load curves where failure took place, some definition of failure was necessary. There seems to be considerable difference of opinion as to what constitutes failure in a concrete beam. Some experimenters do not assume a beam has failed until it breaks in half and falls from its support, others not until a visible crack has appeared, but the Bureau of Public Roads in practically all of its work has called that point failure at which the curve of strain plotted against load begins to change direction rapidly.

In the present instance, owing to the fact that the strain-load curves did not "break" sharply, it was very difficult to determine the strain at which this change of direction took place. Therefore it was decided to adopt that point at which the tangent to the curve would be a line with a natural slope of three-fourths (0.00015 inch per inch ÷ 2,000 pounds). This more or less arbitrary slope was chosen because it seemed to correspond in the majority of cases to loads creating a rise in the neutral axis of the beam

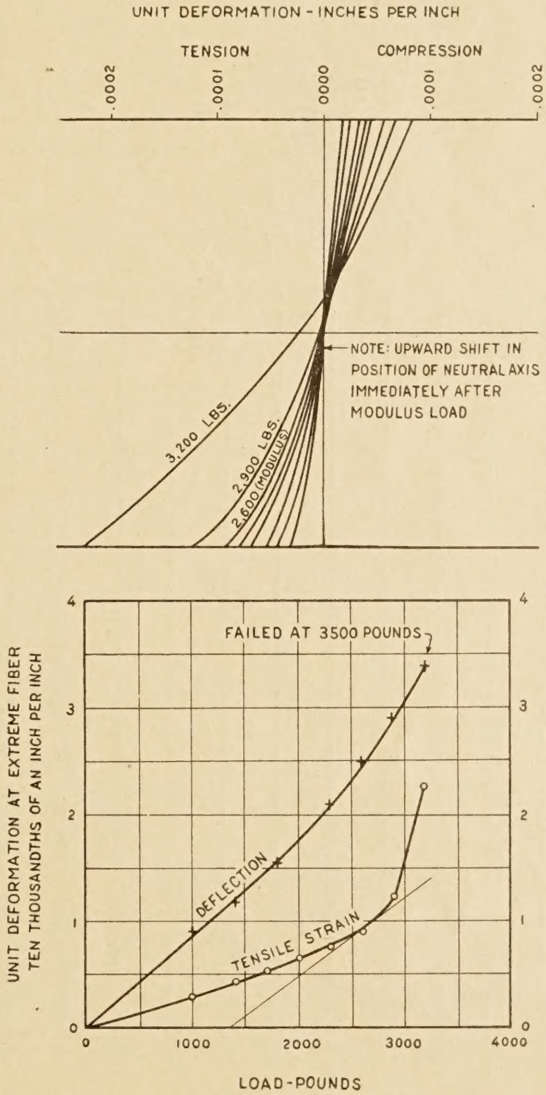

Fig. 6.—Static Test of Beam A-4

and a sharper upward trend in the deflection-load curves. Figures 6 and 7, which are typical, show how the modulus of rupture was obtained; and data resulting from the static and "single-drop" impact tests are presented in Table 1.

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HIGH IMPACT MODULI REDUCED BY REPETITION OF BLOWS

Where the mass was dropped but once from each height it appears that the average modulus of rupture under impact using the new cushion was 35 per cent higher than under static load, and 63 per cent higher when the worn cushion was used. (Figs. 8 and 9.)

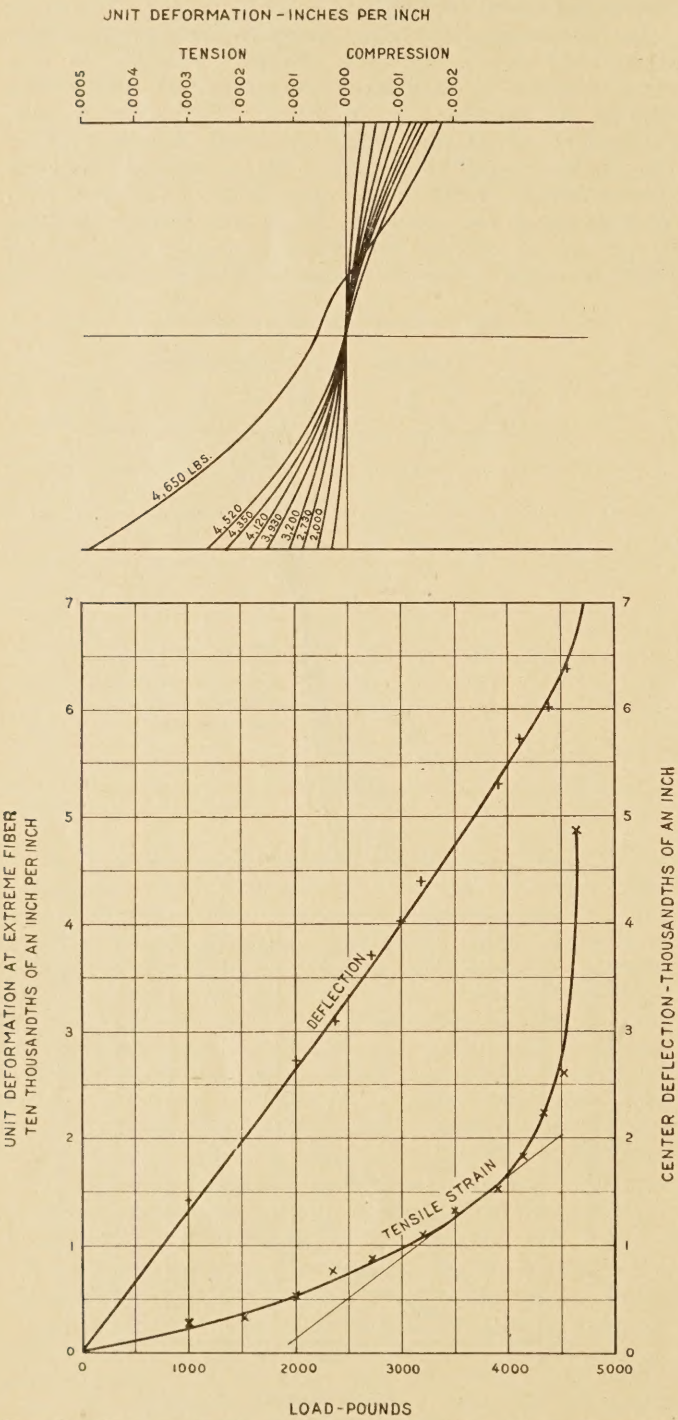

Fig. 7.—Impact Test of Beam A-11 with New Cushion

This conclusion is of little practical value, however, because of the influence upon modulus of rupture of such factors as duration of load and fatigue under repetition. In other words, later tests show that if the load application is repeated a number of times at each successive height it is possible to produce failure at much lower deformations than where the single-drop method is employed.

The explanation of this lies in the fact that it takes time for any material to fail. When the strain is increased by increments a critical stage is reached below which any number of repetitions is successfully resisted and above which one application of sufficiently long duration or a large number of short duration will cause failure, each application contributing toward complete and final destruction.

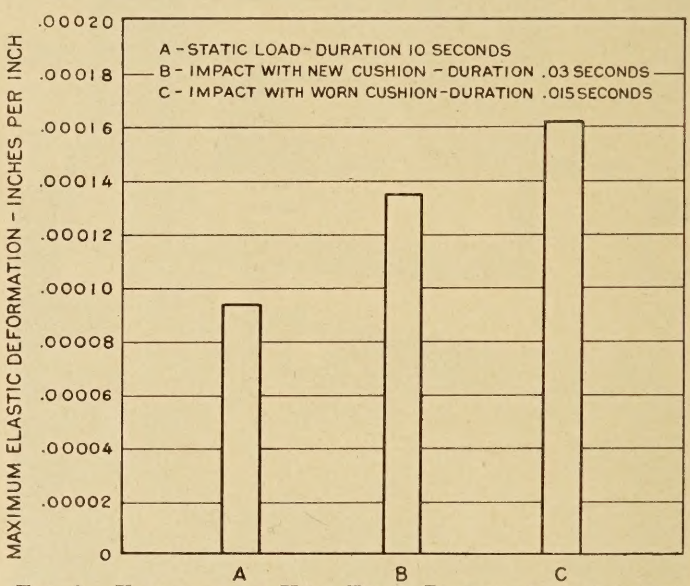

Fig. 8.—Variation of Unit Fiber Deformation for the Three Conditions of Test

Table 1.—*Data resulting from static and "single-drop" impact tests*

(Age of 1:1½:3 concrete 220 days)

Type of test	Beam No.	Ultimate breaking strength of control cylinders	Modulus of elasticity of control cylinders	Fiber deformation corresponding to modulus of rupture in single-drop tests
		Pounds per square inch	*Pounds per square inch*	*Inches per inch*
Static	1	4,410	4,500,000	0.000100
	2	4,430	4,600,000	.000105
	3	4,172	5,050,000	.000090
	4	4,360	4,600,000	.000095
	5	4,330	4,850,000	.000095
	32	4,045	4,450,000	.000082
	33	3,465	4,400,000	.000082
Average				.000093
Impact—new cushion	7	4,231	4,600,000	.000130
	8	4,175	4,550,000	.000125
	9	4,017	4,800,000	.000162
	10	4,226	5,650,000	.000150
	11	3,793	4,400,000	.000140
	26	3,864	4,900,000	.000100
	27	3,929	4,600,000	.000145
	28	3,564	4,200,000	.000115
	29	3,677	4,300,000	.000130
	30	3,944	4,350,000	.000140
	31	3,650	4,650,000	.000140
Average				.000134
Impact—worn cushion	13	4,062	4,600,000	.000180
	14	3,839	4,850,000	.000185
	16	3,304	4,350,000	.000140
	17	4,003	4,600,000	.000170
	19	3,942	4,500,000	.000130
	20	4,110	4,800,000	.000170
	21	4,045	3,850,000	.000190
	22	3,725	4,300,000	.000140
	23	4,062	4,750,000	.000150
Average				.000162

Accordingly, we should expect to find the static load-deformation curves swinging upward at a comparatively low modulus value when we consider that the time of application was about 10 seconds. The concrete was given a relatively long time in which to accomplish partial failure under each load beyond the critical stage referred to. No doubt had some load considerably below the modulus load been permitted to remain on the beam for a very much longer time than 10 seconds, the beam would have failed and we would have had a much lower value of the modulus of rupture. Or again had the static loads above the critical stage been repeated a great number of times the beam would have failed with accompanying low modulus.

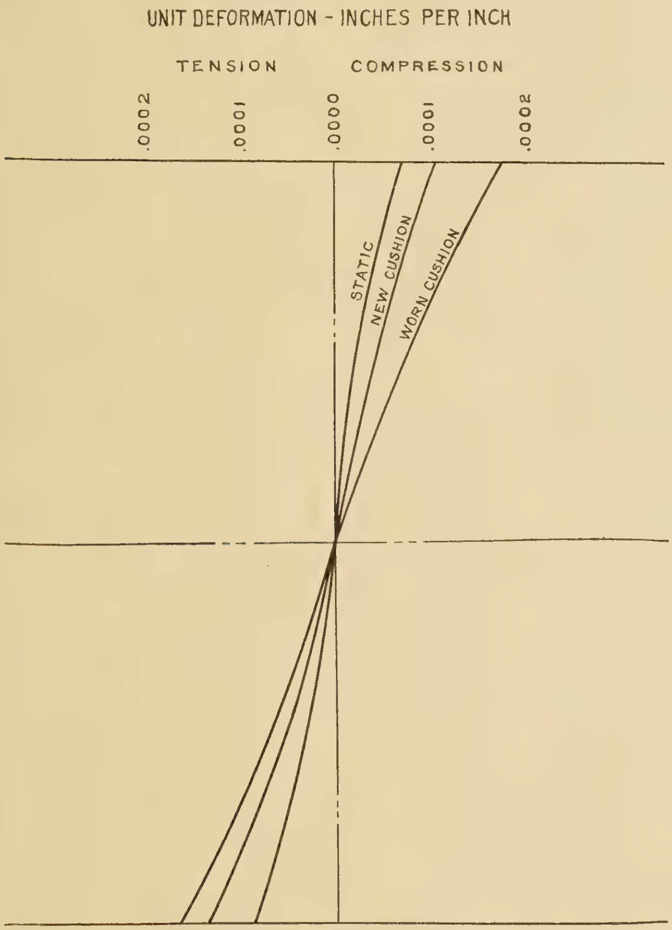

Fig. 9.—Diagram Showing Strain Distribution for the Three Conditions of Test at Modulus Loads

By the same argument the impact deformation-load curves should begin to change direction at a relatively high modulus value because the time of application in this case was only a few hundredths of a second and the concrete was given but little opportunity to fail under any blow. However beyond the critical stage had any application been repeated a great many times we ought to find the beam failing under these repetitions, and this is exactly what takes place in the impact-fatigue tests.

The composite strain-load curves (fig. 10) demonstrate this partial failure very well, although it should be noted that since they were obtained by averaging a great many results they are not particularly accurate, especially at the high values. The curves for the three conditions run along at about the same general slope (a given load causing the same strain regardless of how it is applied) until a value of strain (0.00005 inch per inch) is reached, beyond which the impact fatigue tests tell us we may expect things to happen. After this there is a rapid departure of the static curve from the other two due to the appreciably longer duration of the static loads. The "new cushion" curve, owing to its faster action (duration about 0.03 seconds) continues at the original slope for a while and then begins to swing upward while the "worn cushion" curve holds to the original slope longest because it is the fastest acting (about 0.015 seconds).

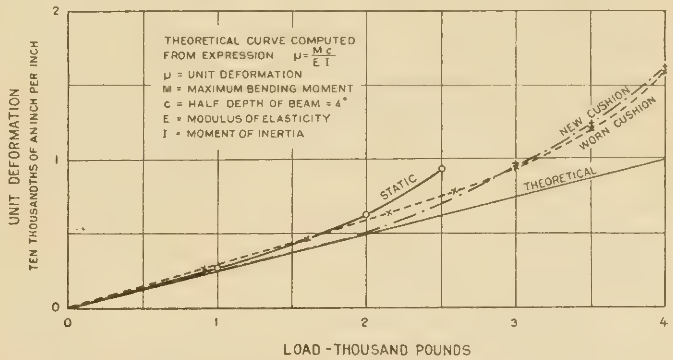

Fig. 10.—Composite Load-Deformation Curves Developed from All Tests

It is felt that the foregoing dicussion explains why such high moduli of rupture were encountered in the impact tests of slabs referred to at the beginning of this report.

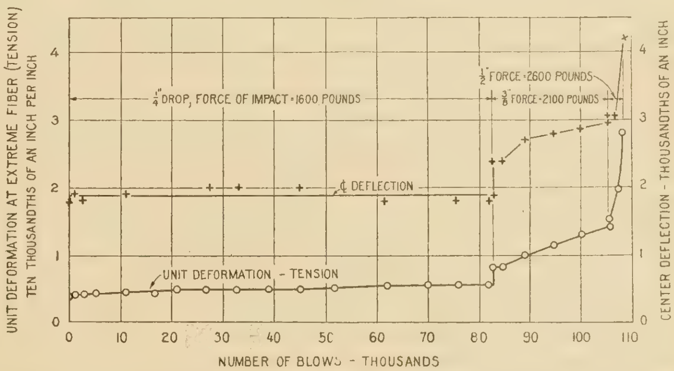

Fig. 11.—Impact Fatigue Test of Beam A-36. The Age at the Beginning of the Test was 277 Days; at the End, 307 Days

IMPACT FATIGUE TESTS

Toward the last of the series four beams were tested in impact fatigue, the rate of loading being approximately 60 blows per minute. Tests were carried on about three hours each day thereby allowing a 21-hour rest period between loadings. This method was employed in an attempt to show the effect of repetition of stress.

The results are shown by the curve (fig. 11) which is typical of all four impact fatigue tests in its essential features. It shows very clearly that for drops of one-quarter inch, producing a strain of 0.00005 inch per inch (corresponding unit fiber stress about 250 pounds per square inch) the beam can be subjected to a very large number of impacts without deleterious effects. As soon as this height is increased to three-eights inch, producing a strain of 0.00008 inch per inch slowly progressing disintegration sets in resulting in final failure.

Since the number of impacts successfully withstood at a strain of 0.00005 inch per inch, in this instance 83,000, is considered to be conservatively representative of any condition under which road slabs are used, the conclusion is reached that this is the upper limit of safe values to which concrete of this mix and age should be subjected.

It is interesting to note, in this connection, that the American Society for Municipal Improvements in a recently issued report of its committee on "Proposed Specifications for Portland Cement Concrete Pavements" recommends an allowable working stress of 250 pounds per square inch for 1:2:4 concrete. It also recommends that "in hot weather, the pavement shall be closed to traffic for at least fourteen (14) days, * * *." As has been pointed out, these tests show that where the pavement is to be subjected to impact

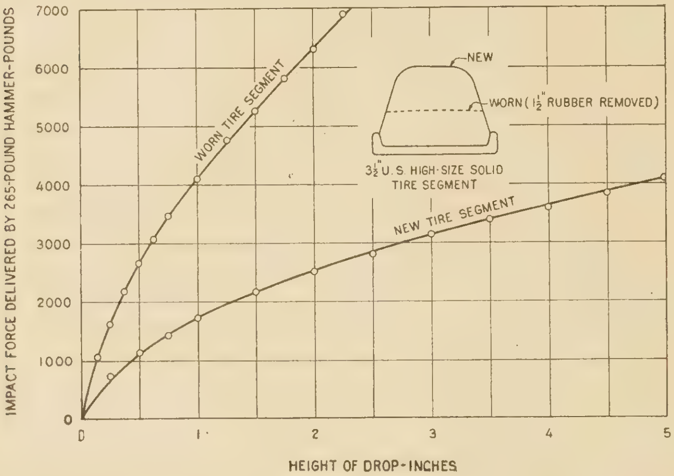

FIG. 12.—CURVES SHOWING EFFECT OF TIRE WEAR ON IMPACT FORCE

fatigue, 250 pounds per square inch is the ultimate safe value for 1:1½:3 concrete, laboratory mixed and carefully cured for 300 days. Considering that concrete trebles its flexural tensile strength between 14 and 300 days,[7] it is interesting to speculate upon what might happen to concrete pavements built according to this specification.

It should be especially noted that the fatigue limit referred to above (strain 0.00005 inch per inch) is 54 per cent of the modulus of rupture obtained statically (strain 0.000093 inch per inch). It may therefore be concluded that under impact loading the same fatigue-limit ratio is obtained as has been shown to exist under static load conditions by the latest fatigue tests.[8]

An inspection of the strain-distribution diagrams (figs. 6 and 7) reveals very clearly the distortion of planes under bending. However, for low values of strain no very considerable error is made in the common assumption that they are not distorted.

The well-known increase in impact force produced by worn as against new tires is clearly demonstrated in the curves. (Fig. 12.) This shows the great disadvantage of using worn tires or, since the use of worn tires can hardly be controlled, the necessity of smoother pavements. The ratio of force produced by the worn tire to that caused by the new tire is about 2.5 throughout the range of comparison.

[7] "Fatigue of Concrete," H. F. Clemmer, Proc. A. S. T. M., vol. 22, Part II, 1922, p. 415.

[8] "Fatigue of Concrete," W. K. Hatt, Proc. Fourth Annual Meeting, Highway Research Board, National Research Council, December 1924. Professor Hatt determined the limit to which concrete might be stressed an indefinite number of times as 55 per cent of the stress at which it failed under progressive static loading.

TRANSPORTATION SURVEYS BEGUN IN NEW HAMPSHIRE AND VERMONT

Highway transportation surveys covering the entire States of New Hampshire and Vermont were begun on July 16 under cooperative agreements between the Bureau of Public Roads and the two State highway departments.

As planned, the surveys are to run for three months, with 13 recording parties taking data twice during the period at each of 143 stations in New Hampshire and 12 parties recording the traffic at 130 stations in Vermont.

In general the same methods will be used as in the Connecticut and Cook County surveys, reports of which have been published, and the Pennsylvania and Ohio surveys which are now being analyzed. For passenger vehicles the data will show the number of vehicles passing the station during each hour, the State in which they are licensed, the number of passengers they carry, whether the purpose of travel is business or pleasure, whether passengers are from city or farm, whether the travel is an extended tour or a short trip, the origin, destination, length of trip, and the number of miles of travel within the State.

For trucks the data will show also the number passing each hour, the State of registration, capacity, origin, and destination, with information as to the character of operation of the consignor and consignee, the total distance of travel and mileage within the State, the commodity transported, the type of trucking (for hire, contract hauling, etc.), and the situs of ownership. In addition to this information there will be recorded at special weight stations the make of truck, type of tires, and weights on front and rear axles.

Analysis of this information will make it possible to prepare maps showing the present volume and distribution of passenger vehicle and truck traffic on the highways of each of the States, from which the State highways will be classified as industrial, high, medium, or low type routes, taking into account motor-truck capacities and loads. A forecast of traffic for several years ahead will be made and a definite program of highway construction worked out. Special attention is to be given to an analysis of the relative traffic importance of the township roads with respect to the primary and secondary systems.

Economic data to be collected will include information concerning the tonnage shipped by motor truck, marketing methods, zones of truck operation, and general trucking practice.

EFFECT OF SIZE OF BRICK ON RATTLER LOSS[1]

Reported by F. H. JACKSON, Engineer of Tests, United States Bureau of Public Roads

IT HAS long been recognized that for brick of equal quality but differing in size the comparative rattler losses are not directly proportional to the differences in weight. Therefore, any system of rating based on the percentage of loss by weight, irrespective of the difference in size, is incorrect unless a correction is introduced covering this feature or unless independent standards are set up for each size separately.

In the very elaborate series of tests presented by Blair and Orton before the American Society for Testing materials in 1911,[2] upon which our present standard method of test is largely based, no mention is made of the effect of size on rattler loss. This was probably due to the fact that at that time the bulk of the paving brick manufactured were of the so-called "standard block" size—that is, about 3 to 3½ inches in width, 4 inches in depth, and 8 to 9 inches in length. Professor Talbot, however, in his paper on "Qualities of High Grade Paving Brick," published in Bulletin 9 of the Illinois State Geological Survey,[3] states that, although he has not studied the effect of the size of brick on the rattler loss, "it is established that the brick size will sustain a greater loss than the block size of the same grade and quality. * * * The amount of this difference depends upon various conditions, but with good material the brick sizes may be expected to lose, say, 3 per cent more than the block sizes."

It has only been within comparatively recent years that the thinner brick, such as those made to lay to a depth of 3 inches and 2½ inches, have come into general use. Efforts have also been made from time to time to introduce certain odd sizes, such as the 3 by 3½ by 8½ inch, and others in which the length varied slightly. The number of sizes of brick in use, however, has been greatly curtailed within the last three years through the efforts of the permanent committee on Simplification of Variety and Standards of Paving Brick of the Department of Commerce. At the present time this committee recognizes two sizes of brick, as follows: 3½ by 4 by 8½ inches and 3 by 4 by 8½ inches. In addition to these two sizes, the 2½ by 4 by 8½ inch size is coming into rather general use, so that it will in all probability in the near future be included in the series of recognized sizes.

Recognizing the injustice of specifying the same percentage of wear for both 3-inch and 3½-inch brick, many paving engineers, when they began using the thinner brick for construction, adopted the practice of inserting certain arbitrary correction factors in their specifications so as to bring these sizes into line with the requirements for the so-called "standard block" size. So far as the writer is aware, however, none of these correction factors was based upon extensive test data. As a rule, they were the result of theoretical consideration.

In view of the fact that committee C-3 of the American Society for Testing Materials has undertaken to rearrange the standard specifications and methods of test for paving brick, C 7–15, in order to bring it into conformity with the society's present standards as to form, it seemed an excellent opportunity to investigate this relationship experimentally with a view to furnishing a table of correction factors which might be inserted in the standard.

TESTS MADE ON FIVE SIZES OF BRICK

Fortunately, a rather unusual opportunity existed for obtaining such data. The Bureau of Public Roads has had under way for the last several months an investigation to determine the relation between the depth of the paving brick wearing course and the resistance of the pavement to the action of traffic. This investigation is being conducted by the bureau in cooperation with the National Paving Brick Manufacturers Association, which furnished a large quantity of brick for this purpose. These brick were all from the same plant and were manufactured as a special lot so as to be as nearly uniform in quality as possible. They were of the plain wire-cut type, and the average rattler loss on the 3½ by 4 by 8½ inch was about 17 per cent. Five sizes of brick were furnished, all of the same width and length but varying in depth from 2 to 4 inches by ½-inch steps.

It was decided, therefore, in addition to the major investigation, the results of which will be reported later, to make an incidental study of the effect of size on rattler loss, using the five sizes of brick on hand. For this purpose 10 standard rattler tests were made on each size of brick, making 50 tests in all. Every precaution was taken to keep the standard rattler calibrated, and every detail as called for in standard procedure was followed. The results for the first series of rattler tests are given in Table 1. Upon studying the results it became immediately apparent that there was no constant relation between the average percentage of loss and the size of the brick. This at once suggested the possibility that the various sizes of brick might not all be of the same quality. In order to throw light upon this important point, a number of brick of each size were subjected to a special hardness test, using the Dorry hardness machine for testing rock. One-inch cores were drilled with a diamond drill from the center of each brick, and subjected to the abrasive action of quartz sand fed upon a revolving steel disk upon which the brick core was held under a standard pressure. The loss in weight of the specimen at 2,500 revolutions of the disk was considered to be a true measure of its hardness.

Realizing that hardness is not the only quality of a paving brick which affects the rattler loss, tests for crushing strength and transverse strength were also made. The results of these tests, together with the results of the hardness tests, are plotted in Figure 1. Crushing strengths were determined on half brick, tested on edge, five tests of each size being made. The brick were bedded in plaster of Paris before testing. Tests

[1] A paper presented by the writer at the annual meeting of the American Society for Testing Materials, June 22 to 25, 1926, Atlantic City, N. J.

[2] "A Study of the Rattler Test for Paving Brick," M. W. Blair and Edward Orton, jr., Proc. A. S. T. M., Vol. XI, 1911, p. 776.

[3] Illinois State Geological Survey Bul. 9, "Paving Brick and Paving Brick Clays of Illinois."

for transverse strength were made in two ways, (1) by using a special form of equalizer apparatus developed at the Bureau of Standards, and (2), by the use of the A. S. T. M. standard apparatus somewhat modified by the Bureau of Public Roads. Details of the results of the transverse tests, discussed from the standpoint of comparative methods of testing, will be published elsewhere. For the purpose of this discussion, the results of the tests by both methods were averaged. Each average is the result of 40 tests.

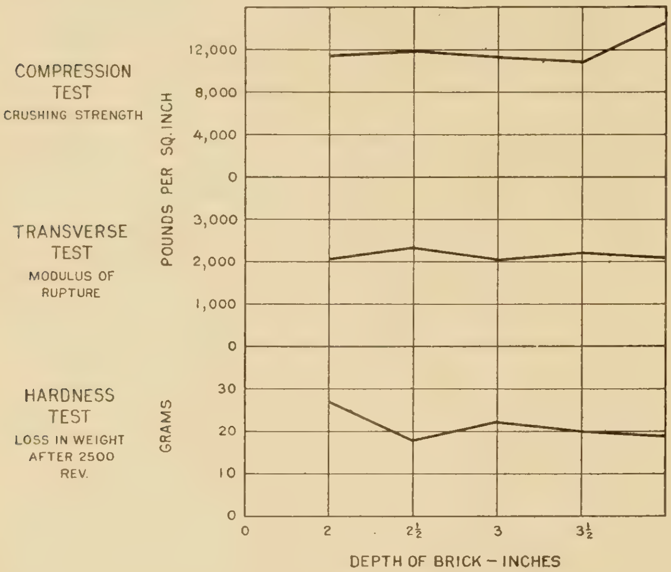

Fig. 1.—Results of Hardness Test, Transverse Test, and Compression Test on Bricks Varying in Depth from 2 to 4 Inches

Table 1.—*Rattler test results on brick used to determine effect of size*

Sample No.	Percentage of loss in weight				
	Thickness of brick (inches)				
	2	2½	3	3½	4
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	22.6	17.9	18.6	17.4	17.3
2	22.0	21.4	18.3	16.8	16.5
3	23.1	18.3	19.7	16.8	16.4
4	21.9	18.3	19.7	16.5	16.3
5	22.8	19.1	18.7	16.2	17.0
6	22.8	19.3	19.7	17.0	17.1
7	22.7	19.0	18.6	17.2	16.3
8	22.9	17.6	19.3	17.0	17.2
9	24.7	19.2	19.1	18.0	17.4
10	23.3	17.9	18.2	17.4	16.6
Average	22.9	18.8	19.0	17.0	16.8
Maximum	24.7	21.4	19.7	18.0	17.4
Minimum	21.9	17.6	18.2	16.2	16.3

A very brief study of these tests is in order with a view to determining what differences in quality exist between the various sizes. It will be seen at once that the tests are not altogether consistent. For instance, the 4-inch size has a considerably higher crushing strength than any of the other sizes. This difference, however, is not reflected in any of the other tests, which is, of course, not surprising when we consider that each of these tests measures a specific property of the brick. When taken as a whole, however, the results give a general idea of the relative quality of the five sizes under consideration which, in this case, is what we are after. Assuming for the moment that the resistance of brick in the rattler is influenced by both hardness and toughness and that these qualities are measured individually probably better by the hardness test and the transverse test than by any of the others, it is found that from the standpoint of hardness the 2-inch brick are considerably softer than any of the other sizes; the 3-inch are next; and the 2½-inch are the hardest. In transverse strength the 2½-inch size ranks highest; the 3½-inch next; with the 2-inch, 3-inch and 4-inch practically identical. Taking both tests into consideration, the brick may be tentatively rated relatively as to quality about as follows: 2½-inch, 3½-inch, 4-inch, 3-inch, 2-inch. It should be borne in mind, of course, that these differences are not large numerically, and are of significance only because of the special use to which the rattler tests will now be put.

EFFECT OF SIZE ON RATTLER LOSS

In Figure 2 are plotted the average losses in pounds for each size of brick against the initial weight of the brick charge. There are also plotted two series of points, one above and the other below the actual losses, which show what the losses would have been if they had been (1) directly proportional to the number of linear inches of edge exposed to wear, and (2) directly

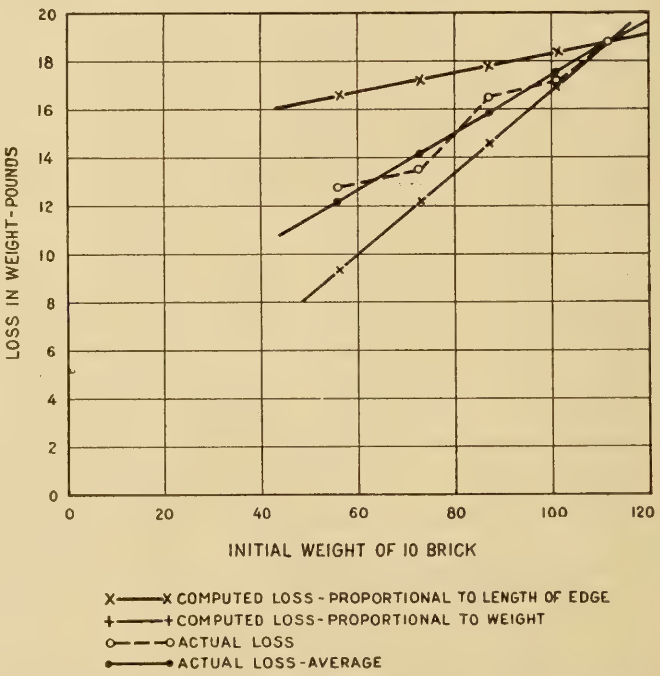

Fig. 2.—Diagram Showing Actual Loss and the Average Actual Loss of Various Sizes of Brick in the Rattler Test. There is Also Shown What the Losses Would Have Been Had the Loss Been Proportional to the Linear Inches of Edge Exposed to Wear and to the Weight of Bricks

proportional to weight (or volume). It will be seen that the actual curve is somewhat below a line bisecting the angle formed by the two theoretical curves, which indicates that the correction to be applied to the small sizes is somewhat smaller than has been commonly used on the assumption that the loss takes place principally on the edges and corners of the brick.

Returning now to a discussion of the actual losses, it is found that the plotted points do not lie on a straight line as they would were the brick all of the same quality and the differences in loss due entirely to the effect of size. Plotting the average line, it is found that the losses for the 2½-inch and 3½-inch sizes lie below the line, whereas those for the 2-inch and 3-inch sizes lie above the line. This grouping is exactly what would be expected as a result of our study of the relative quality of the five sizes based on the hardness and transverse tests, and indicates that these tests are a reliable measure of those properties of the paving brick which are affected by the rattler test.

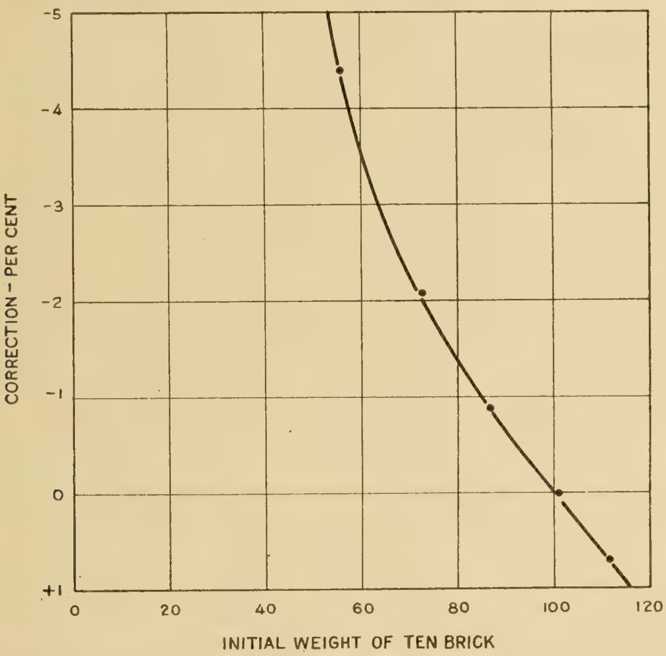

Fig. 3.—Diagram Showing Percentages to be Added to or Subtracted from Observed Rattler Test Loss to Give Results in Terms of "Standard" 10-Pound Brick

One would seem justified, therefore, in drawing the average line as indicated on the chart, and recomputing the various percentages of loss from the corrected losses obtained therefrom. This gives a series of values the same as would have been obtained experimentally had all the brick been of the same quality. Using these values, a correction curve has been plotted as shown in Figure 3. By means of the chart it is possible to determine what percentage shall be added to or subtracted from the observed rattler loss to give the equivalent value in terms of a "standard" 10-pound brick. The corrections, it will be observed, are based entirely on weight and not on nominal size. Although it might be more convenient to use the latter method, it would seem unwise to do so because of the wide variations in actual size of brick of the same nominal size. For instance, among eight brands tested by the bureau in connection with this work, it was found that for the 2½-inch size the actual weights of 10 brick varied from 64 to 75 pounds, whereas for the 3-inch size the corresponding variation was from 82 to 89 pounds. These differences, of course, are accounted for in part by differences in specific gravity of the material. In general, however, the differences in size appear to be more pronounced, as will be observed by noting Figure 4, in which are plotted the average weights and corresponding volumes for each of the brands and sizes of brick tested.

It will be observed that the points lie fairly well on a straight line, those above the line indicating the lighter brick, whereas those below the line indicate the heavier brick. Of course, a correction based on the volume of brick would be the most rational method because it would eliminate variations in the volume-weight relations owing to differences in specific gravity. However, the weight determination is much simpler and more readily made, and it would not appear that the small changes in specific gravity which normally occur in well-burned paving brick would seriously affect the accuracy of the corrections.

SUPPLEMENTARY CHECK TESTS

In order to determine to what exent the proposed correction curve could be applied in actual work, a series of check tests was run on a number of different brands of paving brick submitted by the manufacturers at the request of the National Paving Brick Manufacturers Association. Each brand was represented by at least two sizes, and in some cases by three sizes. Control tests for hardness and for modulus of rupture were likewise run, in order to check the quality of the brick by tests independent of the size factor. Unfortunately, the number of brick available for these supplementary tests was so limited that it was found impossible to obtain a sufficient number of tests for hardness and modulus of rupture to obtain representative averages.

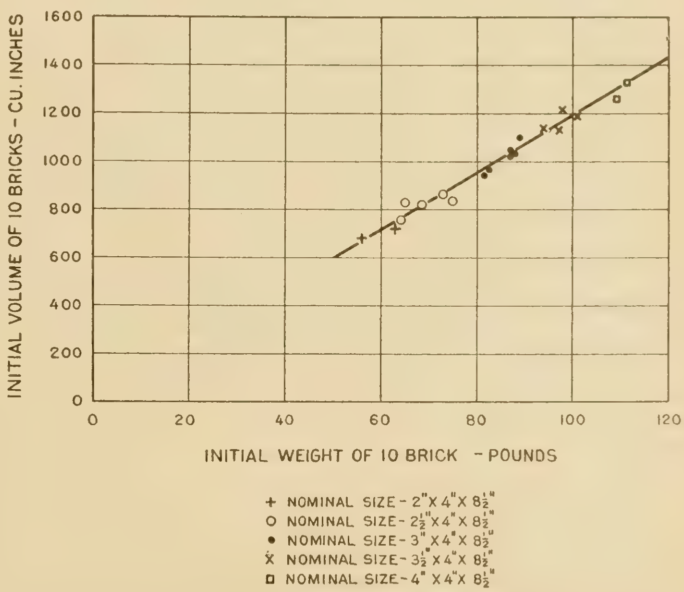

Fig. 4.—Diagram Showing Variation in Weight and Volume of Groups of Ten Brick of Various Sizes and Brands Prepared for the Rattler Test

The results of the rattler tests on six lots of brick, representing four brands of wire-cut-lug, shale brick and two brands of plain, wire-cut, fire-clay brick are shown in Table 2, together with their initial volumes and weights and the corrected percentage of loss derived from the use of the correction curve shown in Figure 3. Each value for per cent of wear represents the average of three tests. It will be observed that in

(Continued on page 107)

A DEVICE FOR MEASURING PRESSURE USED IN MOLDING CEMENT MORTAR BRIQUETTES

Reported by F. H. JACKSON, Engineer of Tests, and D. O. WOOLF, Junior Materials Engineer, U. S. Bureau of Public Roads

THE pressure exerted by the operator in molding the standard 1:3 cement mortar briquette probably has as great an influence on the tensile strength as any other single factor. Notwithstanding this fact, the present standard methods of testing Portland cement do not provide for the application of any definite pressure, the matter being governed by general reference only. This naturally has resulted in considerable variation in the technique employed by different operators and probably has accounted in part, at least, for the wide divergence in strength tests frequently reported when identical samples are tested in different laboratories.

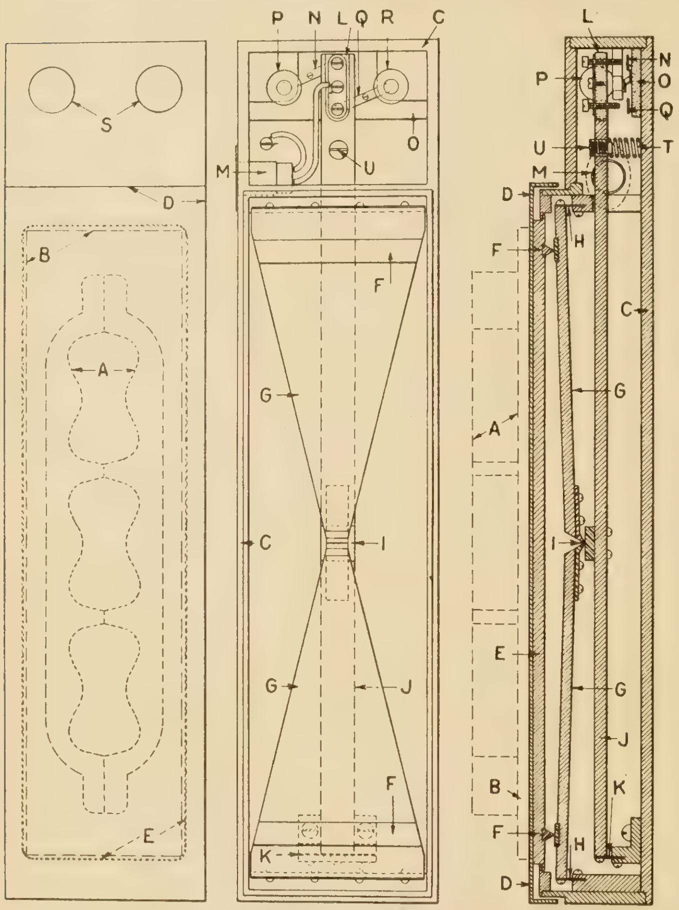

Fig. 1.—Detailed Drawing of Pressure Registering Apparatus

Recognizing the desirability of controlling this as well as many other details of procedure subject to personal interpretation, the American Society for Testing Materials prepared and published a year ago a "Manual of Cement Testing," in which the various methods of testing were described in much greater detail than in the specification proper. In that portion of the manual referring to the molding of briquettes, a pressure of 15 pounds applied to the surface of the briquette was suggested. This value was taken by the committee from the recently proposed master specification for Portland cement of the United States Government and was probably suggested originally by the United States Bureau of Standards. Practically the same pressure, modified so as to call for a minimum of 15 pounds and a maximum of 20 pounds, is provided for in the new tentative specification for Portland cement of the American Society for Testing Materials, adopted provisionally at the June, 1926, meeting. This specification, if adopted by letter ballot of the society, as it probably will be, will automatically become the new American Society for Testing Materials standard some time before the close of the present calendar year.

In the new specification, the requirements for tensile strength of briquettes have been raised from 200 pounds per square inch to 225 pounds per square inch at 7 days and from 300 pounds per square inch to 325 pounds per square inch at 28 days. This increase in the strength makes it all the more important that the technique of testing, in so far as those factors which influence the strength are concerned, be very rigidly controlled. The new values represent more nearly the true tensile strength of many normal Portland cements than the old, so that incorrect methods of testing, which tend to give low results, will undoubtedly cause more rejections. Then too, it is well recognized that many operators, especially those of long experience, have become accustomed to using a pressure considerably greater than that provided for in the new specification. Under the proposed specification, these operators will be required to use a pressure of from 15 to 20 pounds, which will undoubtedly result in lower strengths being reported, still further reducing the factor of safety.

PRESSURE MEASURING INSTRUMENT DESIGNED

Anticipating the need of a simple yet fairly accurate device for measuring the pressure exerted in molding briquettes, the Bureau of Public Roads has designed an instrument for this purpose.[1] The instrument consists essentially of a platform to support the briquette mold, resting upon a multiple lever system which operates against a small coil spring. Two contact points are so arranged on the lever arm that the degree of compression of the spring is indicated by the closing of one or both of two lamp circuits. The initial compression of the spring may be varied so that pressures of from about 10 to 40 pounds may be registered.

A detailed drawing of the instrument is shown in Figure 1, reference to the various parts being indicated by letter. In this drawing, *A* represents a three-gang briquette mold resting upon a glass plate, B, which in turn rests upon the platform of the instrument, *D*. The pressure exerted is transmitted through the knife

[1] The original design of this apparatus was prepared by E. B. Smith, formerly engineer of tests, Bureau of Public Roads.

edges and hardened bearings, *F*, mounted on the transmitting levers, *G*, which have steel ribbon supports, *H*, and steel knife edge points *I*, resting upon the operating lever, *J*. This lever has a steel ribbon support, *K*, at one end, and at the other end, a fiber insert, *L*, carrying three screws, the center one connected by wire to a standard electrical connection, *M*, of dry battery, the other wire being grounded to the case. When the minimum allowable pressure has been applied, one of the remaining screws makes contact with a spring, *N*, mounted on a fiber block, *O*, lighting a white electric lamp, *P*. When the maximum allowable pressure has been reached, the other screw comes in contact with a similar spring, *Q*, lighting a red electric lamp, *R*. Both lamps may be observed by the operator through the holes in the cover, *S*. A coiled spring, *T*, adjusted by the screw, *U*, returns the operating lever to its initial position when the pressure is released.

The instrument is calibrated by placing a static load, including the weight of the briquette mold, etc., upon the platform, equivalent to the minimum pressure it is desired to register. The adjusting screw over the spring connected to the white lamp is then turned until contact is just made. An additional load, making the total equivalent to the maximum allowed, is then placed upon the platform and the screw over the spring connected with the red lamp is similarly adjusted.

This device has been in operation in the laboratories of the Bureau of Public Roads for several months and has proved very satisfactory. It should be possible to regulate the applied pressure by means of this device to any specified value plus or minus two or three pounds. In other words, such tests as the bureau has made would indicate that, by the use of the instrument, it is possible to mold briquettes conforming to the new standard specification in which the minimum pressure is specified at 15 pounds and the maximum pressure at 20 pounds.

EFFECT OF VARYING PRESSURE ON THE STRENGTH OF BRIQUETTES

To determine the variation in strength resulting from molding briquettes at different applied pressures, a series of tests was recently conducted, using several different pressures. Two series of test specimens were made, with molding pressures and number of applications to each briquette face as follows:

Pressure, pounds	Number of applications per briquette face	Pressure, pounds	Number of applications per briquette face
15–20	12	15–20	18
20–25	12	20–25	18
25–30	12	25–30	18
30–35	12	30–35	18

Tests were made at the age of 7 and 28 days. The two series were then repeated with a second brand of cement and tests made at the age of 7 days. Each average is the result of 12 breaks. The average results of these tests are shown in Table 1.

TABLE 1.—*Effect of pressure and number of applications on average tensile strength of cement*

Applied pressure, pounds	Number of thumbings per briquette face	Tensile strength—pounds per square inch		
		Cement A		Cement B
		7 days	28 days	7 days
15–20	12	245	365	280
20–25	12	250	370	285
25–30	12	260	370	290
30–45	12	260	370	290
15–20	18	260	370	285
20–25	18	260	370	300
25–30	18	265	390	290
30–35	18	270	385	300

Considering the several lots of 12 specimens each, it will be observed that the average tensile strength increases in general with increase in the applied pressure and with the number of applications of pressure. This would, of course, be expected. It should be noted, however, that the increase in strength is not quite so large as has commonly been supposed. For instance, tests at 7 days, using 12 applications with a 15 to 20 pound pressure, gave in the case of the first cement an average of 245 pounds per square inch, whereas a

FIG. 2.—MOLDING BRIQUETTES, USING THE APPARATUS FOR CONTROL OF PRESSURE

pressure of 30 to 35 pounds gave an average of 260 pounds per square inch, or an increase of 15 pounds. The corresponding variation in strength at 28 days was only 5 pounds, the 15 to 20 pound pressure giving an average of 365 pounds and the 30 to 35 pound pressure an average of 370 pounds. Likewise, increasing the number of applications of pressure from 12 to 18 gave an increase in strength of approximately 10 pounds per square inch for both periods. Similar variations may be noted in the case of the second cement. In general it was found that increasing the pressure from about 15 to 30 pounds resulted in an increase of strength of from 10 to 15 pounds, the greater divergence being at 7 days, whereas an increase in the number of thumbings for each briquette face from 12 to 18, resulted in an average increase of strength of about the same amount.

In Table 2 are shown the mean as well as the maximum deviations of individual breaks from the average of 12. It will be noted that in so far as concordance of results is concerned, there is little to choose between the various pressures tried. The same comment applies to the tests made with 12 applications of pressure as compared with those made with 18 applications. It will be noted that in many cases the maximum deviation from the average exceeded the 15 per cent allowed by the specification in routine testing.

Detailed working drawings of the instrument described in this paper have been prepared and will be furnished upon request.

TABLE 2.—*Deviation of individual test results from the average values*

Applied pressure, pounds	Number of thumbings per briquette face	Deviation from average tensile strength					
		Cement A				Cement B	
		7 days		28 days		7 days	
		Mean	Maximum	Mean	Maximum	Mean	Maximum
		Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
15–20	12	6.8	14.3	8.7	16.4	4.4	8.9
20–25	12	4.7	10.0	5.4	17.8	4.8	12.3
25–30	12	5.2	15.4	6.0	14.9	7.1	12.1
30–35	12	5.4	9.6	6.0	16.2	4.5	8.6
15–20	18	4.5	13.5	5.9	13.5	5.5	14.0
20–25	18	4.8	15.4	4.9	13.5	3.9	6.7
25–30	18	5.8	13.2	4.4	11.5	4.4	6.9
30–35	18	5.1	14.8	6.2	19.5	3.8	8.3

THE STRENGTH OF MORTAR AND CONCRETE AS INFLUENCED BY THE GRADING OF THE SAND

Reported by J. G. ROSE, Materials Engineer, United States Bureau of Public Roads

IN CONNECTION with the testing of materials for Federal-aid highway projects in Colorado a study has been made of the relation between the grading of sand for use in concrete and the strength developed in mortar and concrete; and the study has led to the development of a graph which may be used as the basis for a preliminary judgment of the quality of sands proposed for use.

The samples of sand and gravel, or crushed rock, upon which the study was based were contributed by the State highway department of Colorado as materials to be tested for use on Federal-aid projects. Approximately 200 samples of sand and gravel are represented by the study. The source of the materials was widespread, almost every county in the State having contributed one or more samples.

The testing work was done by the Pierce Testing Laboratory, of Denver, and standard methods of testing concrete materials approved by the American Society for Testing Materials were followed in making all tests. Standard briquettes of 1:3 mortar were used for the tensile tests, and 6 by 12 inch cylinders of 1 : 2 : 4 mix were used for the compression-test specimens. The consistency of the concrete was such as to show a slump of from 1 to 2 inches as determined by the standard slump-cone method. All observations are based upon the 28-day strength of the specimens, both in tension and in compression.

While assembling the test data for the study it was observed that the strength of the 6 by 12 inch cylinders varied from about 1,500 pounds per square inch to a little over 3,500 pounds per square inch. In order to observe the variation in grading between the high and low strengths, the test reports were divided into four groups, each group having a range in strength of 500 pounds; and after computing the average grading of the maximum strength group (3,000 to 3,500 pounds per square inch) and plotting this average grading upon the graph, it was observed that part of the reports in the remaining three groups fell above the maximum strength curve, and part fell below it. The three groups were, therefore, divided again into two groups each, depending on whether the grading of the samples fell above or below the maximum strength curve. Samples falling partly above and partly below the maximum strength curve were listed in both groups; and the average gradings for the six groups thus obtained were then computed and plotted on the graph. The groups thus established, according to grading, were then averaged for tensile strength.

WHAT THE GRAPH SHOWS

The curves derived for the above averages as shown in Figure 1, lead to the following conclusions:

1. That there is an ideal grading of sands which will produce maximum strength in concrete.
2. That the ideal grading curve assumes an arched form showing a predominance of the material retained upon the coarser sieves.
3. That for a given mix, there is a practical limit to the quantity of material passing each size sieve, where a given strength of concrete is required.
4. That an exceptionally high tensile strength of sand in 1:3 mortar is not necessarily associated with a high compression strength of the same material when mixed with the average coarse aggregate in concrete, hence the tensile strength is not a proper gauge of the quality of a sand for concrete.

Justification of the relation between the grading and strength of sands as shown by the curves is dependent upon a combination of coordination of several well known factors or theories of concrete. The maximum strength curve is, doubtless, associated with maximum density, or minimum voids, in the combined aggregates. The decline in strength as the grading becomes finer is associated with increased surface area, and a corresponding increase of voids. As the grading of the sands becomes coarser than that shown by the maximum strength curve, the probability of increased strength indicated by the surface-area theory is over-come by a tendency of the coarser grains of sand to wedge themselves in between the coarse aggregate, thus increasing the voids to such an extent that a deficiency of mortar is produced. Hence, a decline in compression strength is recorded with the increase in coarseness of the sand. This, at least, is one explanation which comes to mind. Probably there are others. The relation of the grading curves to the tensile strength developed in 1:3 mortar bears out the surface-area theory; that is, that the finer the sand the

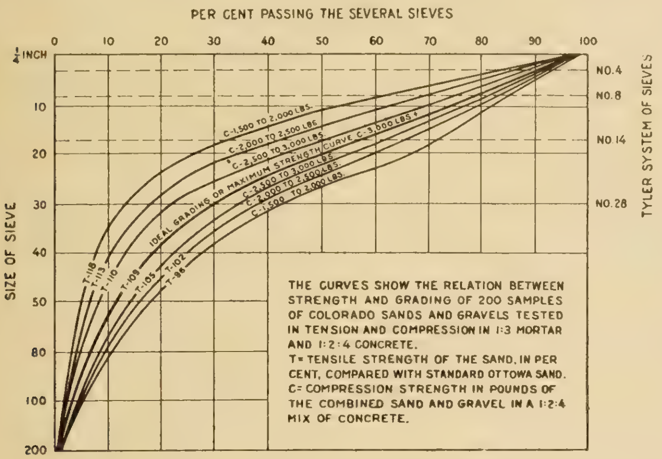

Fig. 1.—Sand Analysis Chart

greater will be the surface area to be covered by a unit volume of cement, hence, the weaker the bond.

UTILITY OF THE GRAPH

By plotting the sieve analysis of a sand upon the graph, a ready means of visualizing the quality of the material for concrete is produced. The area between the upper and lower curves on the graph forms a practical safey zone for the grading of acceptable sands. If the plotted grading of a sand falls outside of this area, in whole or in part, there is but little chance that it will pass standard specification requirements without increasing the proportion of cement.

In making a materials survey for a project, selection of the best source of supply will be greatly facilitated by a comparison of the mechanical analyses of the samples when plotted on the graph. As a precaution in making selection of a sand, it should be realized that several other factors in addition to the grading affect the strength of sands. Variation in structure and soundness, and the presence of organic matter, silt, clay, acids, alkali, and other foreign substances all have their influence on the strength of the sand in concrete. Hence, a considerable variation in strength from the average strength curve for each group shown on the graph should be expected. Eliminating these factors the range of grading for any given strength and mix should be small. Final selection, of course, should always be determined by a more complete laboratory test. But once a complete laboratory determination for the quality of a sand has been made, and the mechanical analysis plotted, any change in quality due to variation in grading is easily detected by a screen analysis made in the field during the progress of construction and plotted on the graph.

(Continued from page 103)

Table 2.—*Check tests of paving brick*

Lot No.	Nominal size	Volume of 10 brick	Weight of 10 brick	Rattler loss	Corrected loss	Total variations within lots: Before correction	Total variations within lots: After correction
	Inches	*Cubic inches*	*Pounds*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	3	1,009	87	24.2	23.3		
	2	720	63	28.1	24.8	3.9	1.5
2	3½	1,219	98	19.9	19.8		
	3	1,113	89	21.3	20.6	1.4	.8
3	3½	1,128	97	16.1	15.9		
	3	1,028	87	16.7	15.8		
	2½	838	75	17.8	15.9	1.7	.1
4	4	1,266	109	17.0	17.5		
	3	974	83	17.9	16.7		
	2½	762	64	18.3	15.2	1.3	2.3
5	3½	1,141	94	17.1	16.8		
	2½	831	65	21.0	18.0	3.9	1.2
6	3½	1,156	94	19.7	19.3		
	2½	811	64	23.2	20.1	3.5	.8

all cases but one the differences in percentage between the corrected losses for the different sizes in any given lot are considerably less that the corresponding differences before correction. In only one case, however, that of lot 3, have these differences entirely disappeared, indicating that a certain amount of the original variation was due to differences in the quality of the brick. This is not surprising when it is remembered that no special effort in any case was made to obtain brick of exactly the same quality. In one case, that of lot 4, the results of the individual rattler tests were very erratic, indicating an extremely nonuniform product, which may account for the wide variations of the results obtained on this lot.

It is felt that the correction curve obtained as a result of this series of tests is sufficiently accurate for all practical purposes; and the writer has recommended that a table of correction factors based thereon be considered for use in connection with any revision of the present standard rattler test for paving brick. The following addition to paragraph 13 of the standard specifications for paving brick has been suggested:

The result obtained in the original calculation shall be corrected by adding to or subtracting from it a factor depending upon the initial weight of the brick charge in pounds. This factor shall be obtained from the following table:

Original weight of 10 brick	Correction to be applied to percentage of wear obtained by test	Original weight of 10 brick	Correction to be applied to percentage of wear obtained by test
Pounds	*Per cent*	*Pounds*	*Per cent*
105 to 115	0.5+	75 to 79	2.0−
95 to 104	.0	70 to 74	2.5−
90 to 94	.5−	65 to 69	3.0−
85 to 89	1.0−	60 to 64	3.5−
80 to 84	1.5−	55 to 59	4.0−

The final corrected value, together with the observed value and the correction factor, shall be reported.

MORE ACCURATE TESTS OF REINFORCING BARS

Reported by D. O. WOOLF, Junior Materials Engineer, U. S. Bureau of Public Roads

IN TESTING steel reinforcement bars, it has been the custom in many laboratories to assume that the cross-sectional area of the bar is that given in the manufacturer's tables. It has been found that, in the case of the deformed bar, the nominal area seldom agrees with the actual area, and that occasionally the difference is so great that serious errors may result from the assumption of equality. To insure more accurate tests of the physical properties of steel, the physical laboratory of the Bureau of Public Roads has adopted the practice of making planimeter measurement of the cross-sectional area of all concrete reinforcement bars submitted for test. The higher allowable unit stresses used in designing at the present time together with a desire for more rigid laboratory control have demanded that this greater care be used in determining the bar cross section.

The method used in the determination of the actual area is simple. The test specimen is cut through at right angles to its axis, and the cut surface is filed smooth. An impression of this surface is made by the use of carbon paper or stamp pad, and the area of the print is determined by polar planimeter measurement. In the case of bars of very irregular shape, several different sections are measured.

The standard specifications for billet-steel concrete reinforcement bars (serial A15–14) of the American Society for Testing Materials permit the testing of bars turned down to a uniform cross section. This method of testing deformed bars has the objectionable feature of requiring the use of a lathe which may not be available at the laboratory. By the use of the planimeter the cross-sectional area may be determined so accurately that the turning down to uniform diameter will not be necessary.

Table 1 shows certain data from some of the more recent tests made by the bureau. The unit tensile strength has been computed using both the nominal and actual cross-sectional areas. In several cases, had the nominal area been used the bar would have been classed as other than its true grade of steel. For example, samples 6 to 8, inclusive, were submitted as ½-inch round bars. Measurement of the cross-sectional area gave 0.25 square inch in each case, and computation of the test values gave unit tensile strengths corresponding to the structural grade. Using the nominal area of a ½-inch round bar, unit tensile strengths in the intermediate grade were obtained. As the specifications for this project required the structural grade, the steel would have been rejected had the nominal area been used.

TABLE 1.—*Comparison of actual and nominal cross-sectional areas of bars and unit strengths based on each*

Sample No.	Size and shape of deformed bar	Area of cross section		Breaking load	Unit tensile strength		Error in nominal unit tensile strength	
		Nominal	Actual		Nominal	Actual		
		Sq. ins.	*Sq. ins.*	*Pounds*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Per cent*
1	½-inch square	0.25	0.20	12,140	48,560	60,700	12,140	20.0
2	⅜-inch round	.11	.16	10,990	99,910	68,690	31,220	45.4
3	½-inch round	.20	.25	15,320	76,600	61,280	15,320	25.0
4	½-inch square	.25	.23	14,590	58,360	63,430	5,070	8.0
5	¾-inch round	.44	.41	24,740	56,230	60,340	4,110	6.8
6	½-inch round	.20	.25	15,620	78,100	62,480	15,620	25.0
7	½-inch round	.20	.25	15,650	78,250	62,600	15,650	25.0
8	½-inch round	.20	.25	15,560	77,800	62,240	15,560	25.0
9	¾-inch round	.44	.43	36,260	82,410	84,330	1,920	2.3
10	1-inch square	1.00	.98	61,820	61,820	63,080	1,260	2.0

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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.

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387D. Public Road Mileage and Revenues in the Southern States, 1914.
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390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
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670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
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1259D. 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.
1279D. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

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331C. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

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59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
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Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D-3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

*Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

JUNE 30, 1926

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2,863,197.86	611.8	$ 12,256,313.63	$ 5,862,787.23	686.5	$ 5,058,420.96	$ 2,374,991.75	207.3				$ 3,248,478.16	Alabama
Arizona	9,580,133.43	5,016,119.94	613.8	1,369,744.82	847,652.41	116.0	1,401,980.83	945,372.27	81.4	$ 82,189.48	$ 50,225.99	12.2	2,757,878.39	Arizona
Arkansas	13,310,190.08	5,380,181.73	1048.9	5,054,354.42	2,286,516.62	274.1	4,758,782.28	2,248,758.94	308.8	389,196.91	193,512.38	31.1	1,496,834.33	Arkansas
California	22,346,175.99	10,719,249.61	894.8	4,796,420.91	2,284,342.69	163.2	11,715,699.34	5,695,750.56	319.9	223,161.51	133,896.90	18.2	3,239,575.24	California
Colorado	11,876,703.94	6,067,814.34	651.2	2,029,200.70	1,059,473.84	93.8	4,456,058.36	3,192,681.79	214.4	726,547.90	326,138.34	21.8	2,679,703.69	Colorado
Connecticut	4,558,639.29	1,819,368.66	101.6	855,927.90	281,217.14	15.5	3,259,571.40	921,363.03	48.0	924,669.87	221,419.88	9.9	1,090,312.29	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	623,535.35	218,310.90	15.7	3,503.60	Delaware
Florida	2,959,273.72	1,405,487.97	96.3	873,406.54	418,874.35	36.6	9,925,038.28	4,645,765.56	279.7				1,614,826.12	Florida
Georgia	20,156,002.37	9,406,366.46	1478.3	4,635,204.60	2,257,871.40	315.7	12,261,322.41	6,036,376.25	656.9	1,438,858.45	690,662.84	28.3	40,676.05	Georgia
Idaho	9,394,676.80	4,815,332.26	600.1	1,666,521.34	1,066,780.44	124.6	2,958,753.24	1,789,391.33	162.5	766,646.86	454,085.19	72.5	434,037.78	Idaho
Illinois	40,010,481.10	18,640,076.28	1236.2	4,106,130.76	1,979,919.46	141.5	6,290,805.18	3,030,099.67	213.5	683,153.78	331,828.09	24.1	5,850,274.50	Illinois
Indiana	13,639,172.65	6,562,455.68	422.1	3,310,253.22	1,609,669.51	112.2	15,956,296.59	7,437,580.19	428.0	1,532,011.45	763,564.44	51.7	1,831,085.18	Indiana
Iowa	27,272,285.21	11,107,492.99	1996.9	1,790,090.19	818,809.11	117.9	9,830,045.68	4,770,528.91	648.7	2,865,214.46	1,329,250.33	146.8	1,459,481.66	Iowa
Kansas	26,399,695.77	9,755,273.32	831.4	6,426,905.87	2,835,215.93	329.2	11,703,240.93	4,616,421.11	651.9	1,867,736.46	676,229.78	129.9	1,581,270.86	Kansas
Kentucky	14,832,324.28	6,205,994.59	584.9	5,905,381.82	2,286,087.66	173.4	5,631,422.97	2,666,402.84	280.0	95,483.33	47,741.66	3.6	2,006,582.25	Kentucky
Louisiana	11,939,424.97	5,279,870.86	927.6	1,891,167.71	864,869.13	128.3	3,435,673.08	1,667,428.72	161.2	875,858.92	383,621.41	29.8	1,076,617.88	Louisiana
Maine	8,174,281.31	3,907,870.33	281.4	573,271.45	284,637.06	22.2	2,364,417.18	854,849.24	67.3	1,344,693.66	555,945.87	45.6	861,525.50	Maine
Maryland	8,132,506.90	3,849,383.15	294.4	2,792,436.20	1,263,608.07	128.9	513,398.28	252,643.43	30.7	1,280,407.48	535,388.21	57.7	24,034.14	Maryland
Massachusetts	14,047,656.22	5,467,661.28	300.6	4,306,101.49	1,189,599.34	73.9	3,548,227.96	996,569.84	46.5	1,389,802.47	337,508.17	21.2	2,117,387.37	Massachusetts
Michigan	16,234,000.80	7,328,316.91	612.6	9,763,239.98	4,498,735.39	350.4	10,129,758.58	4,599,288.88	246.5	208,854.00	99,254.00	11.6	3,816,769.82	Michigan
Minnesota	30,415,685.89	12,738,642.04	2721.2	6,755,300.06	2,847,474.52	460.7	9,330,294.00	3,493,000.00	591.3	443,985.90	31,000.00	11.1	481,663.44	Minnesota
Mississippi	10,292,285.79	4,988,702.73	803.4	4,853,802.73	2,425,831.37	325.6	6,629,582.39	3,283,796.56	360.6	1,916,418.91	867,077.13	95.1	562,610.21	Mississippi
Missouri	17,368,156.57	8,219,411.43	1118.9	11,621,010.35	5,516,603.42	424.3	20,568,938.77	8,128,296.60	563.6	1,215,244.48	516,163.45	23.1	405,961.10	Missouri
Montana	10,156,600.41	5,317,523.15	921.6	1,244,383.40	1,015,942.74	133.3	1,749,936.25	1,194,956.27	132.7	1,770,351.41	1,345,318.41	145.7	4,551,144.43	Montana
Nebraska	9,306,374.36	4,389,523.50	1570.6	2,227,027.26	1,084,679.02	197.7	13,349,102.13	6,510,012.74	1352.1	943,438.61	470,648.61	94.0	2,180,371.13	Nebraska
Nevada	4,917,465.69	3,088,299.78	357.3	2,640,729.82	2,042,634.81	181.5	3,250,322.24	2,798,063.96	346.1	254,858.63	222,534.71	48.4	643,681.74	Nevada
New Hampshire	4,165,687.86	1,986,226.87	208.1	826,870.74	391,223.20	29.5	866,830.33	396,691.78	26.0	232,305.40	104,291.27	6.9	291,058.88	New Hampshire
New Jersey	11,961,357.45	3,820,679.99	219.1	4,384,943.56	1,277,662.22	71.2	6,814,161.67	2,519,262.44	31.6	567,180.59	150,315.00	10.0	699,500.35	New Jersey
New Mexico	8,717,999.18	4,914,070.61	1081.3	3,686,338.59	2,425,586.77	345.7	1,349,543.87	873,541.00	100.4	567,172.13	457,488.45	34.0	2,301,699.17	New Mexico
New York	28,597,769.67	12,229,076.53	831.5	14,626,510.12	5,682,880.66	365.5	32,340,520.00	8,967,680.20	567.4	9,943,103.00	2,201,500.00	136.6	4,964,057.61	New York
North Carolina	21,014,450.41	8,746,454.59	1119.8	5,994,969.06	2,430,883.35	138.1	8,035,569.05	3,484,782.13	190.2	636,619.50	317,809.75	26.1	737,276.18	North Carolina
North Dakota	10,829,263.82	5,268,930.47	1917.5	1,484,047.58	762,929.31	275.6	5,408,990.29	2,744,108.72	750.9	2,954,245.24	1,468,329.42	388.8	504,361.08	North Dakota
Ohio	41,572,252.81	15,244,993.93	1191.1	6,127,280.09	2,126,793.10	173.0	11,554,505.96	4,475,373.81	353.5	2,980,871.01	1,146,320.00	85.8	2,738,315.16	Ohio
Oklahoma	20,787,024.94	9,672,890.34	852.2	7,460,925.39	3,487,108.81	326.7	2,893,119.44	1,354,980.82	97.4	726,614.30	343,264.78	73.1	1,201,542.25	Oklahoma
Oregon	14,388,188.70	7,142,364.63	794.6	2,639,689.72	1,450,850.16	144.6	3,262,909.64	1,794,020.71	126.3	409,330.64	237,616.61	22.0	254,494.89	Oregon
Pennsylvania	43,054,835.19	16,222,023.97	850.3	18,311,315.61	5,338,708.07	338.5	25,913,665.91	7,325,756.54	529.7	2,405,915.24	753,285.81	57.4	1,699,006.61	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	485,392.16	124,365.00	8.3	557,219.94	Rhode Island
South Carolina	11,163,347.84	5,121,267.54	1235.9	3,857,292.06	1,645,055.39	246.0	6,332,252.88	2,883,133.47	240.8	362,029.99	114,921.58	19.3	37,146.02	South Carolina
South Dakota	12,091,434.67	5,989,879.00	1447.9	5,376,938.52	2,613,947.97	733.3	3,716,153.62	1,864,529.03	633.3	551,764.94	299,132.42	119.8	399,301.58	South Dakota
Tennessee	13,789,140.98	6,732,079.77	497.9	7,835,490.59	3,544,504.25	282.1	7,630,941.58	3,490,104.05	248.4	1,513,770.60	628,578.00	24.6	885,324.93	Tennessee
Texas	54,120,970.83	21,057,940.12	3907.1	15,062,702.65	6,382,314.60	1013.1	19,070,379.25	8,428,039.53	907.4	2,418,044.93	1,101,193.14	61.4	3,636,943.61	Texas
Utah	6,259,159.41	3,818,836.91	423.1	1,994,018.62	1,279,603.77	123.3	1,688,956.94	1,269,611.66	154.0	785,929.62	599,447.91	49.4	851,278.75	Utah
Vermont	3,015,174.51	1,452,834.45	107.8	1,226,868.13	564,805.06	26.7	1,837,974.09	723,688.66	34.6	71,000.71	21,322.97	1.5	505,795.86	Vermont
Virginia	13,099,720.01	6,271,998.20	676.2	8,890,529.43	4,113,729.91	329.3	6,468,112.20	2,824,006.82	185.7	401,916.32	189,322.63	10.9	102,456.44	Virginia
Washington	13,352,504.18	6,117,211.87	526.7	3,726,007.45	1,665,697.59	141.9	3,309,677.73	1,653,600.00	41.1	1,171,789.67	414,000.00	45.3	295,266.54	Washington
West Virginia	7,343,200.86	3,230,293.33	326.7	2,130,515.58	910,769.32	66.2	5,291,716.29	2,060,920.93	137.1	1,195,906.96	519,235.61	40.3	631,291.81	West Virginia
Wisconsin	21,807,140.91	8,919,640.62	1451.7	3,049,367.28	1,473,065.11	140.4	6,916,454.17	3,369,475.19	313.4	1,661,466.49	825,731.00	102.8	2,850,903.08	Wisconsin
Wyoming	8,809,819.33	4,739,096.67	982.0	2,118,483.23	1,301,790.38	151.5	2,635,904.29	1,663,408.02	213.7	126,205.97	81,023.00	10.4	780,955.93	Wyoming
Hawaii							1,050,897.93	312,635.18	15.9				787,517.82	Hawaii
TOTALS	740,140,790.82	325,654,346.00	41,898.3	226,552,043.54	100,524,357.58	10628.3	347,113,666.05	148,527,474.03	14355.1	56,029,885.69	22,839,821.04	2483.8	73,769,001.35	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 84,710,263.69 Federal aid $ 37,375,850.32 Miles 3393.8