PUBLIC ROADS

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BUREAU OF PUBLIC ROADS

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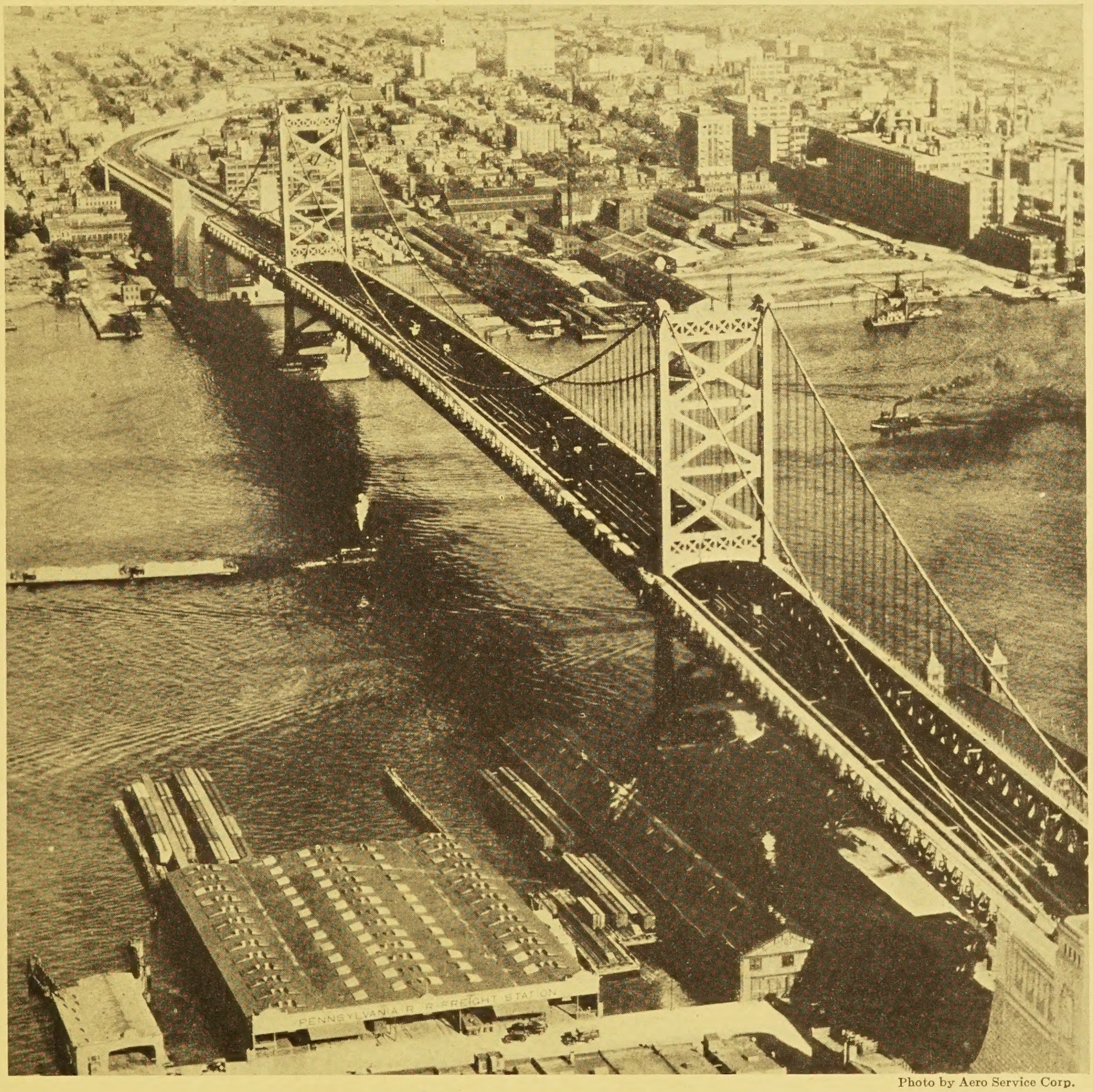

Photo by Aero Service Corp.

THE DELAWARE RIVER BRIDGE AT PHILADELPHIA

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

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TESTS OF THE DELAWARE RIVER BRIDGE FLOOR SLABS

By the Division of Tests, United States Bureau of Public Roads. Reported by GEORGE W. DAVIS, Assistant Engineer of Tests

CONCRETE floor slabs, similar in all respects to those used in the construction of the Delaware River Bridge at Philadelphia and which are of special and unusual design have been subjected to a complete set of tests by the Bureau of Public Roads in cooperation with the Delaware River Bridge Joint Commission. The slabs were designed to reduce to a minimum the weight of the floor system of the Delaware River Bridge which at the time of its opening was the longest suspension span in the world. It has a main span of 1,750 feet and two side spans of about 720 feet each or a total length, including approaches, of 1.8 miles. The main and side spans are supported by two main cables, each 30 inches in diameter, from which are suspended the stiffening trusses which, in turn, support the floor system.

In a bridge of such magnitude it is obvious that the reduction of its dead weight to a minimum is a major feature of design. The structure is primarily a highway bridge and the dead weight is largely that of the highway deck. The design prepared for this portion of the structure, a reinforced concrete slab with a bituminous wearing surface, differed from previous practice to such an extent that tests were undertaken to determine the strength and behavior. These tests have confirmed theoretical conclusions as to the adequacy of the design and yielded data of value in designing similar floor systems. The tests were made at the Arlington Experimental Station of the bureau.

BRIDGE FLOOR OF UNUSUAL DESIGN

The main cables of the bridge are 89 feet apart. From them is suspended a floor system 125 feet 6 inches in width. In the center of this is a 57-foot roadway. On each side of the roadway is space for two electric car lines, one inside and one outside of each of the stiffening trusses, which hang directly under the main cables. Overhead footwalks, 10 feet in width, are carried on cantilever brackets on the outside of the trusses.

The floor system consists of steel floor beams riveted to each panel point of the stiffening trusses, these floor beams being 20 feet 6 inches center to center in the main span and 20 feet 8 inches center to center in the side spans. Longitudinal 18-inch **I**-beam stringers between the floor beams and spaced 3 feet 10 inches on centers support the concrete floor slab. Expansion joints are provided at every second floor beam.

The floor slab rests directly on the stringers. It is designed for a concentrated wheel load of 15,000 pounds with an allowance of 50 per cent for impact. On the suspended spans it is 6 inches in thickness and is covered with a 2½-inch asphalt wearing surface. The transverse or main slab reinforcement consists of Rivet-grip fabricated trusses spaced 6 inches on centers. The chords of these trusses are small rolled channels spaced 4½ inches back to back and the web bars are flats rolled with projecting lugs on the sides. In the process of manufacture the web bars were first bent to the required shape and then assembled with the chord channels and passed through a press which caused the flanges of the channels to grip the lugs of the web bars. The longitudinal slab reinforcement consists of ½-inch round deformed bars spaced 6 inches on centers in the bottom of the slab and 12 inches on centers in the top. The reinforcement has a ¾-inch cover top and bottom. The concrete used was a 1:1½ : 3 mix.[1]

The tests at Arlington were made upon two floor sections of similar design and approximately the same span length but of about one-third the width. The test slabs were supported by floor stringers resting upon concrete abutments instead of steel floor beams as in the bridge. Details of the test slabs are shown in Figure 1. All materials used in these slabs were identical with those used on the bridge and from the same sources.

The tests as run may be divided into two groups. The tests on the first slab were primarily proofing tests under both static loads and under impact to prove by actual physical tests that the slabs were adequate to withstand any probable loads to which they might be subjected.

The tests on the second slab were made to show the effect of the consistency of the concrete on the strength of the slab and to provide more exhaustive data as an aid to the design of slabs of this type. In this report each slab will be treated separately.

SCOPE OF TESTS ON FIRST SLAB

Materials used.—The aggregates used were Delaware River sand and gravel from above Bordentown, N. J.

The steel used consisted of Rivet-grip 4½-inch reinforcing trusses and ½-inch round deformed bars. (Fig. 2.) Test samples taken from the upper and lower chords of the reinforcing trusses showed a modulus of elasticity of 30,000,000 pounds per square inch.

The cement used showed a tensile strength of 315 pounds per square inch at the end of 7 days, and 355 pounds per square inch at the end of 28 days. Test cylinders from the concrete for the first slab showed a modulus of elasticity of 4,137,000 pounds per square inch and a compressive strength of 5,465 pounds per square inch at the end of 28 days, while at the end of 235 days a compressive strength of 5,975 pounds per square inch was developed.

Construction.—The slab was cast in a building which has a reinforced concrete floor 12 inches in thickness. This floor is supported on reinforced concrete beams 12 inches square on 8-foot centers both ways, carried on columns at beam intersections. With the slab in place no measurable deflection of the floor could be found by measurement with an Ames dial reading to 0.0001 inch, under a centrally applied static load of 30,000 pounds. The steel **I** beams under the test slab rested on concrete abutments 1 foot thick and 3 feet high built directly on the floor.

The mix used was 1:1½ : 3. The average slump was 2.9 inches using a 4 by 8 by 12 inch cone, and the average flow was 115.

[1] A more complete description of the floor system is contained in an article entitled "Reinforced Concrete Bridge Roadway on the Delaware River Bridge," published in Concrete, vol. 30, No. 2, February, 1927.

The concrete was mixed in a power-driven mixer for not less than two minutes, wheeled to the forms in wheelbarrows, dumped to a platform, and shoveled in place in the forms. The mixture was flowed into place, and air forced out by vibrating forms and reinforcement with an electric hammer provided with a flat-nosed bit and delivering 3,600 blows per minute. The slabs were covered with wet burlap immediately after the placing was completed. The forms which were wet before the concrete was placed were removed at the end of 24 hours. The burlap and the bottom of the slab were kept wet for seven days. The slab was 28 days old before any loads were applied to it.

During the tests on the first slab the following measurements were made:

a. Deflection of the slab and the **I** beams under static loads.
b. Deformations in the slab and in the **I** beams under static loads.
c. Deformations in the reinforcing trusses under static loads.
d. Spread of the two center **I** beams under static loads.
e. Deflection of the slab under impact.
f. Deformations in the slab and in the **I** beams under impact.

TEST INSTRUMENTS DESCRIBED

Static loads.—All static loads were applied at the center of the slab with a 150-ton hydraulic jack, which rested on a cast-iron block, which in turn rested on a rubber cushion consisting of three segments of a 30 by 4 inch solid rubber tire.

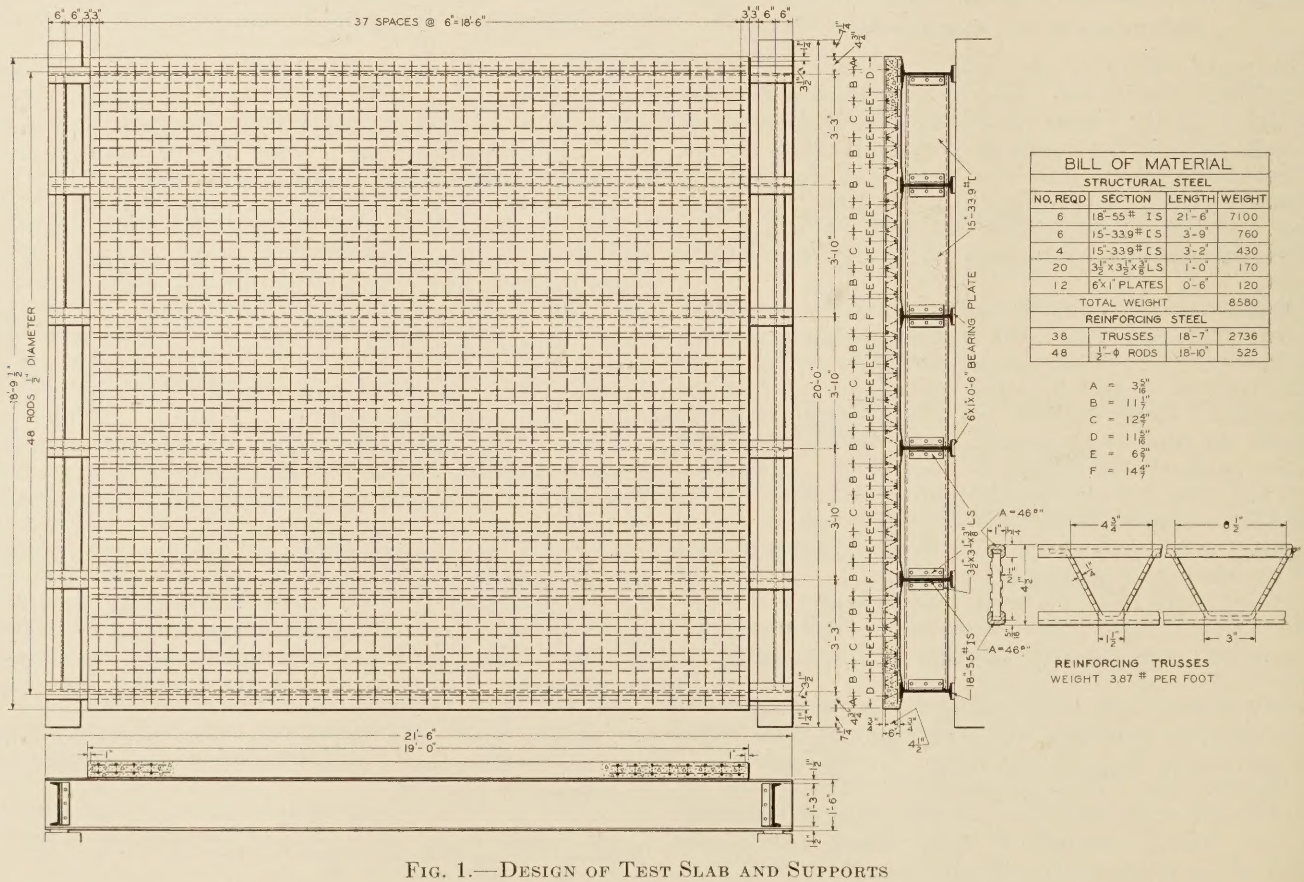
These tire segments were set in the direction of traffic (parallel to **I** beams). The jack reacted through an iron strut against two overhead **I** beams, 18 inches in depth, which were anchored to the foundations of the building by suitable steel work. The magnitude of the load was determined through the measured deflection of a calibrated beam. This deflection was measured with a 0.0001-inch Ames dial, the stiffness of the beam being such that one division on the Ames dial indicated a load of 185 pounds. Freedom of movement of the rider bar (fig. 3) on the calibrated beam and proper contact of the stem of the Ames dial was insured by vibration of the rider bar with an electric buzzer as the load was applied. Details of the static loading device are shown in Figures 3 and 4.

Deflections.—Deflections of the slab were determined by measuring the distance between gauge points on the bottom of the slab and **I** beams and corresponding points fixed on the concrete floor beneath with a wooden staff provided with a metal shoe at the lower end and an Ames dial at the upper end. The gauge points on the slab were small brass blocks, in which conical depressions had been drilled. These blocks were cast in place in the slab during construction.

The gauge points in the **I** beams consisted of conical depressions drilled in the **I** beams. The fixed points on the floor were similar conical depressions drilled in strips of 1 by ¼ inch steel bolted and grouted to the floor. A pair of steel blocks with like conical depressions were set into the concrete abutment at a fixed distance apart to serve as a standard to permit a check of the staff during the progress of the measurements. The layout of deflection points is shown in Figure 5.

Deformation and stress.—The deformations in the concrete and **I** beams were measured with 6-inch graphic strain gauges. This small device [2] has a system of levers so arranged as to magnify any movement of the gauge points at its ends about 75 times and

[2] More detailed information regarding this gauge and its operation may be found in the Engineering News-Record of Mar. 29, 1923, p. 575.



BILL OF MATERIAL			
STRUCTURAL STEEL			
NO. REQD	SECTION	LENGTH	WEIGHT
6	18"-55# I S	21'-6"	7100
6	15"-33.9# [S	3'-9"	760
4	15"-339# [S	3'-2"	430
20	3½"x3½"x⅜" L S	1'-0"	170
12	6"x1" PLATES	0'-6"	120
TOTAL WEIGHT			8580
REINFORCING STEEL			
38	TRUSSES	18'-7"	2736
48	½"-ϕ RODS	18'-10"	525

Fig. 1.—Design of Test Slab and Supports

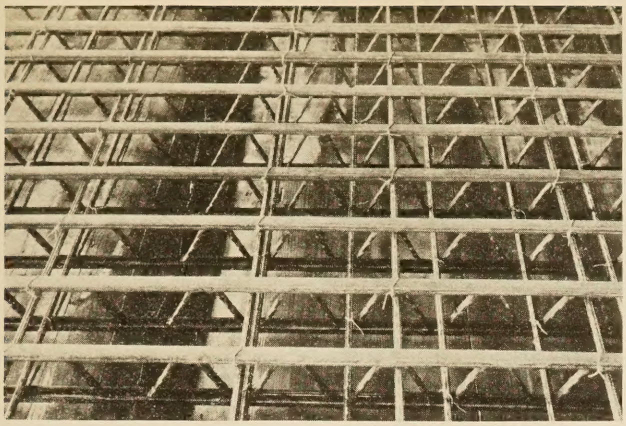


FIG. 2.—REINFORCING IN PLACE FOR TEST SLAB

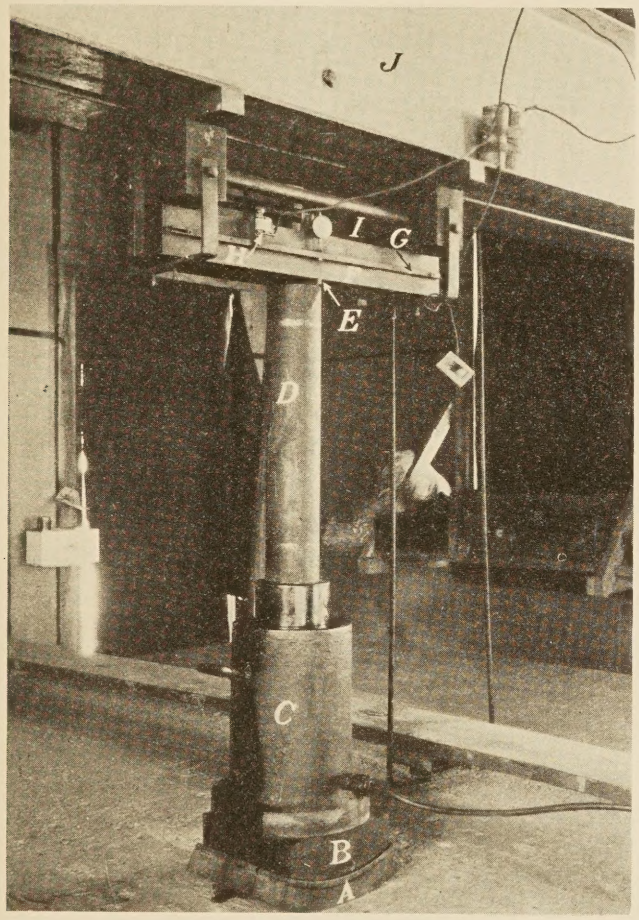


FIG. 3.—STATIC LOADING DEVICE—

A=rubber cushion.	*F*=calibrated beam.
B=cast-iron block.	*G*=rider bar.
C=hydraulic jack.	*H*=buzzer.
D=strut.	*I*=Ames dial.
E=knife-edge.	*J*=overhead I beams.

records the movement on a smoked glass. It can be used to measure both contraction and expansion and the smoked glass can be adjusted to record several successive movements of the gauge points. The graphic record on the glass can be measured with suitable microscope equipment or may be projected on a screen with rectangular coordinates and the actual gauge point movement determined from measurement and the known constants of the instrument and projection apparatus.

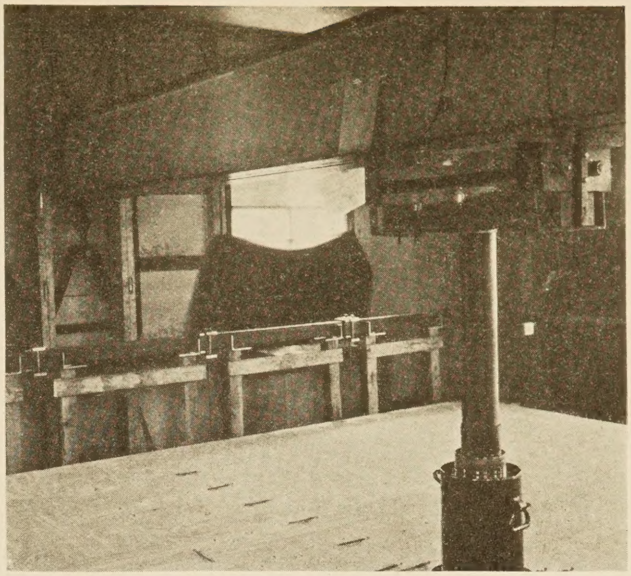


FIG. 4.—APPARATUS USED IN TESTING SLABS. THE DEVICE USED TO DETERMINE I-BEAM REACTIONS CAN BE SEEN AGAINST THE WALL

Gauges were placed on both bottom and top of slab and also on the webs of the **I** beams. The gauges on the slab were set between brass plugs set in the concrete. Those on the **I** beams were located 7 inches above and 7 inches below the axis of the beams and were set between brass plugs attached to the webs of the **I** beams by small bolts passing through the webs. Two 4-inch strain gauges were set at the center points on the bottom of the two center reinforcing trusses, between brass bearing plugs bolted to the lower chords of the trusses, slots being cut through the concrete to the steel. The layout of graphic strain gauges is shown in Figure 6 and some of them can be seen in Figure 4.

Spread of center I beams.—The spread, or the amount that the two center **I**-beam stringers were forced apart at their midspan by static loads, was measured by two wooden rods, each with an Ames dial mounted at one end and placed horizontally between the two **I** beams. Conical depressions were drilled in the webs of these **I** beams at the center of their span 3 inches below the top and 3 inches above the bottom of the beam, or on 12-inch centers. A wooden staff with a metal shoe on one end and an Ames dial on the other was placed between the two upper depressions and another similar staff between the two lower depressions. Both rods were supported by a framework attached to the **I** beam in such a way that they might move with the beams as they deflected.

Impact.—Impact was applied to the center point of the slab with the impact machine which has been used on previous research by the Bureau of Public Roads.[3] This machine consists of a framework of structural steel on which is mounted a truck wheel fastened to a truck spring. Jackscrews at the corners of the frame and an electric motor suitably geared to two cams allow this wheel to be raised and suddenly dropped any desired distance. The sprung weight may be varied by changing the spring tension with two adjusting screws, while the unsprung weight may be varied by increasing or decreasing the weight of the free falling

[3] See Public Roads, vol. 5, No. 2, April, 1924.

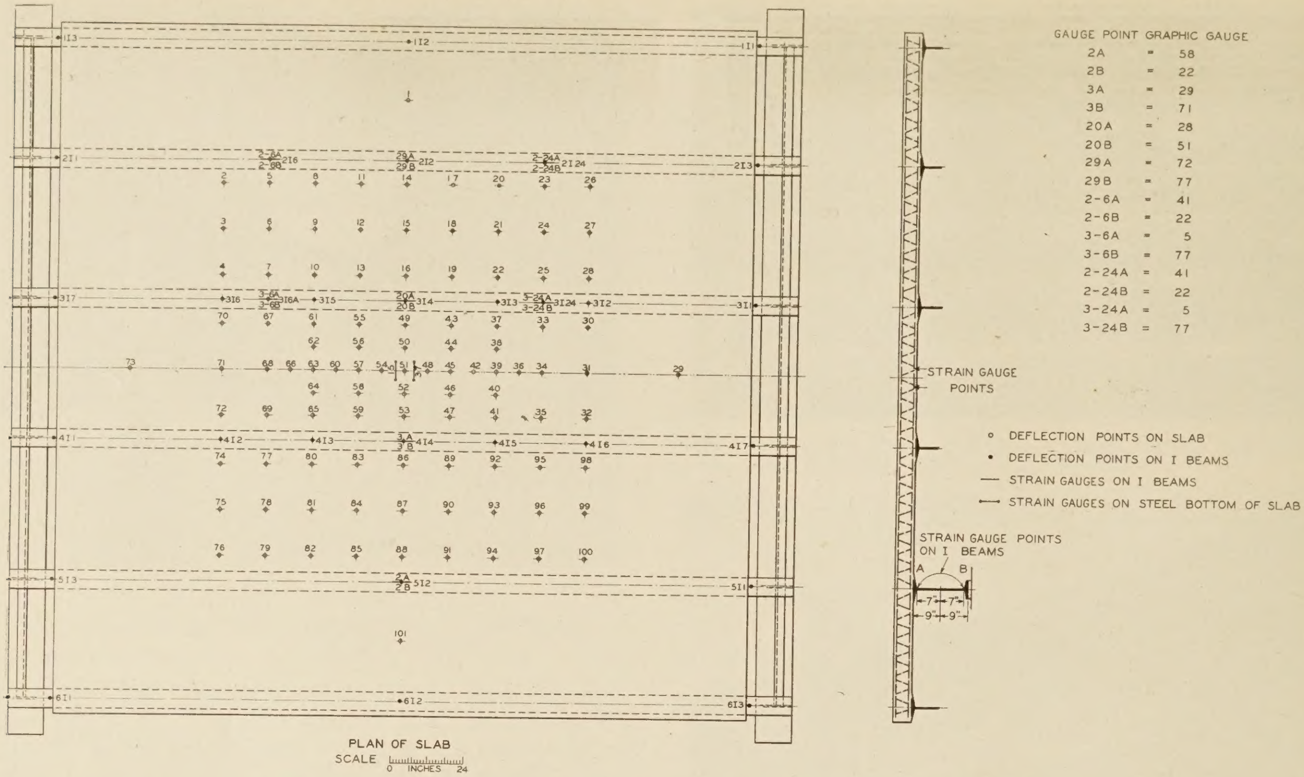


FIG. 5.—LAYOUT OF DEFLECTION POINTS AND GRAPHIC STRAIN GAUGES

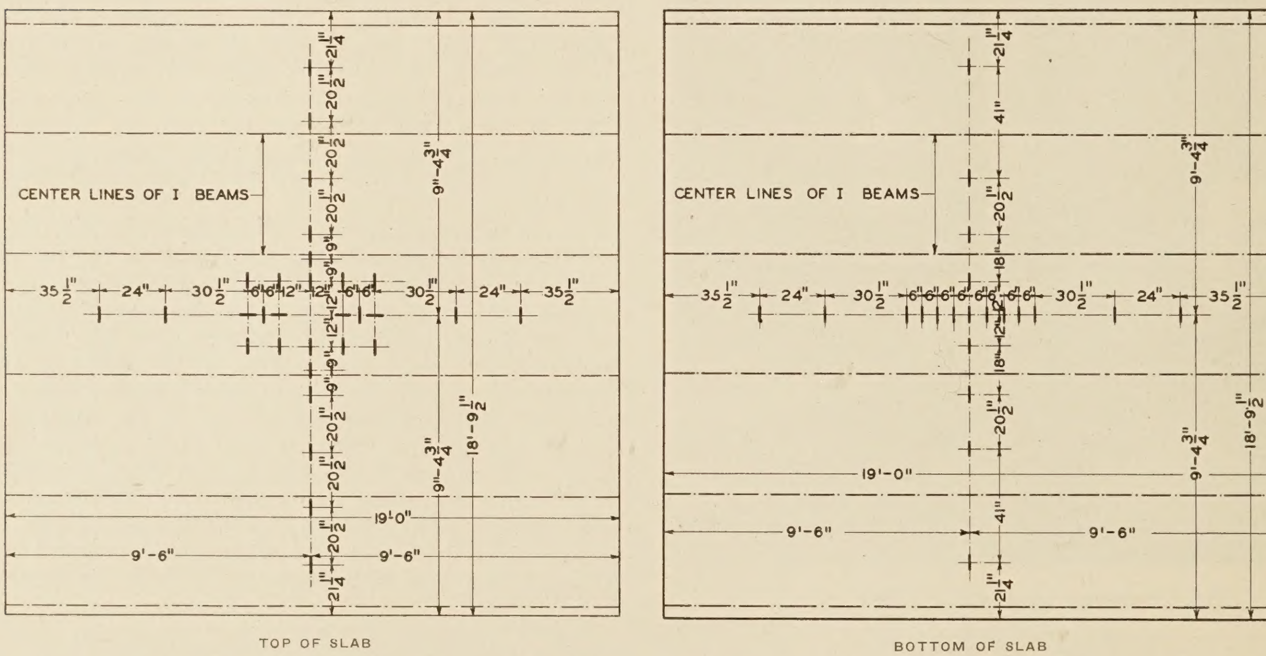


FIG. 6.—LAYOUT OF GRAPHIC STRAIN GAUGES IN TOP AND BOTTOM OF FIRST SLAB

portion of the machine. For these tests a 36 by 6 inch solid rubber tire was used with both a 5-ton Pierce-Arrow truck spring and a 7½-ton Mack truck spring. The magnitude of the impact pressure was determined with a Bureau of Public Roads' coil spring accelerometer.[4] A general view of the impact machine is shown in Figure 7.

[4] See Public Roads, vol. 5, 10, December, 1924.

Deflection under impact.—In the measurement of the deflection of the slab under impact use was made of the same deflection points, both on the slab and the floor, that were used under static loading. Wooden rods with a metal shoe at the lower end and an Ames dial at the upper end were set up at selected deflection points, immediately under and surrounding the point of impact. Around the stem of each of these dials a

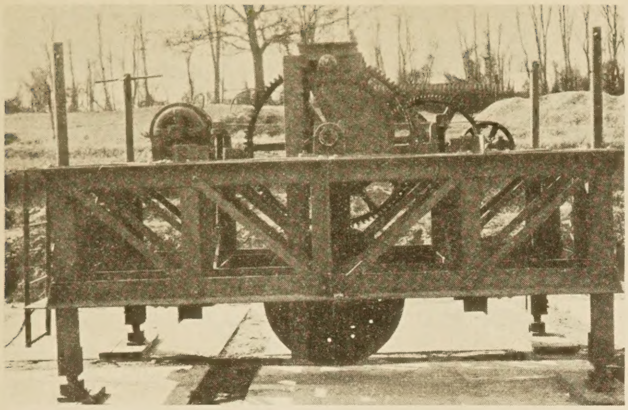
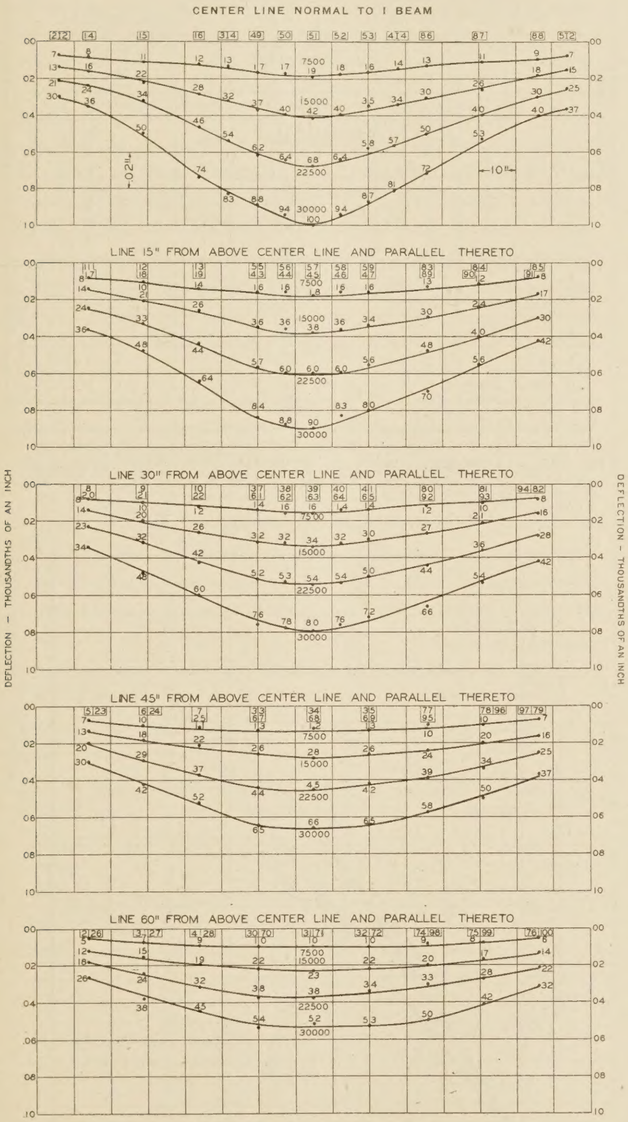


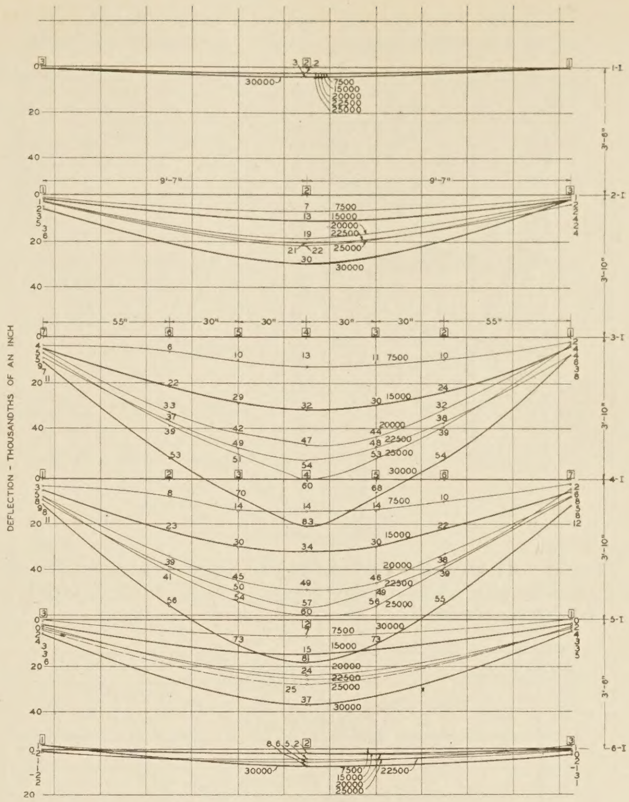
FIG. 7.—THE IMPACT MACHINE USED FOR IMPACT TESTS ON SLAB



[0.0] indicates deflection point

FIG. 8.—DEFLECTION OF FIRST SLAB UNDER STATIC LOADING BEFORE IMPACT

brass collar, fitted with a set screw, was placed. This set screw was tightened sufficiently to overcome, or choke, the action of the spring within the dial, but not to such an extent as to prevent movement of the plunger in the dial under a blow. The rods were set in place and all dials set at zero with the wheel of the impact



[0.0] indicates deflection point

FIG. 9.—DEFLECTION OF I BEAMS FOR 7,500, 15,000, 20,000, 22,500, 25,000 AND 30,000 POUND LOADS APPLIED BEFORE SLAB WAS SUBJECTED TO IMPACT. MEAN VALUES OF DEFLECTIONS SHOWN ON CURVES

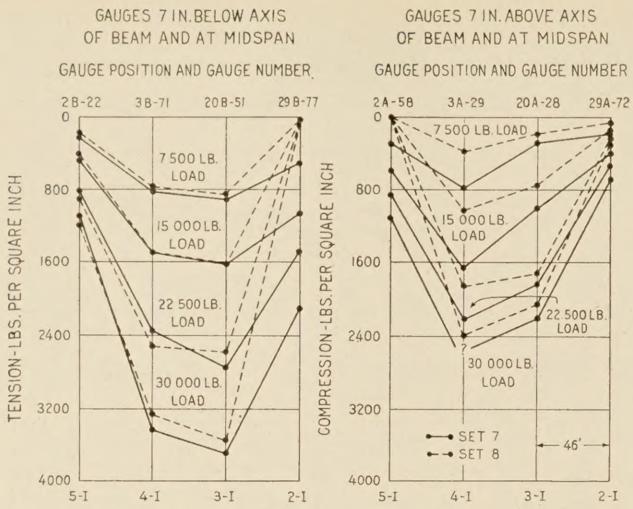
machine on the slab. The wheel was then raised free of the slab and the rise of the slab due to the release of sprung load or wheel pressure was measured by pushing the plungers of all dials up against the slab with the finger. The wheel was then dropped and the deflection of the slab due to the impact blow was read on the choked dials.

TEST PROGRAM ON FIRST SLAB

Static loads in increments of 7,500 pounds up to 30,000 pounds were applied at the center of the slab. Deflection readings and strain-gauge records both in the slab and in the steel were made for each increment. The load was removed by decrements of 2,500 pounds and deflection readings at the center of the slab and at the centers of the two center I beams were taken as a measure of the hysteresis or lag of the slab. After an interval of at least 12 hours, to allow complete recovery of the slab, the same series of loads was again applied and similar data taken. This program was repeated and several sets of readings were secured which agreed within close limits. Within this limit of loading no cracks or signs of distress were apparent.

Mean values of deflection are shown in Figures 8 and 9. Deformations in the slab and stresses in the I beams and reinforcing trusses are shown in Figures 10, 11, and 12. Hysteresis or lag is shown in Figures 13 and 14.

The impact machine was then set up with the wheel over the center of the slab (point 51) and a series of blows delivered. These blows simulated one rear wheel of a 5-ton truck carrying a 2½-ton or a 5-ton load and dropping ½, 1, and 1½ inches. The force of these impact blows was determined by means of the



indicates deflection point

Fig. 10.—Indicated Tensile and Compressive Stresses in I Beams at Gauge Points as a Result of Static Loads

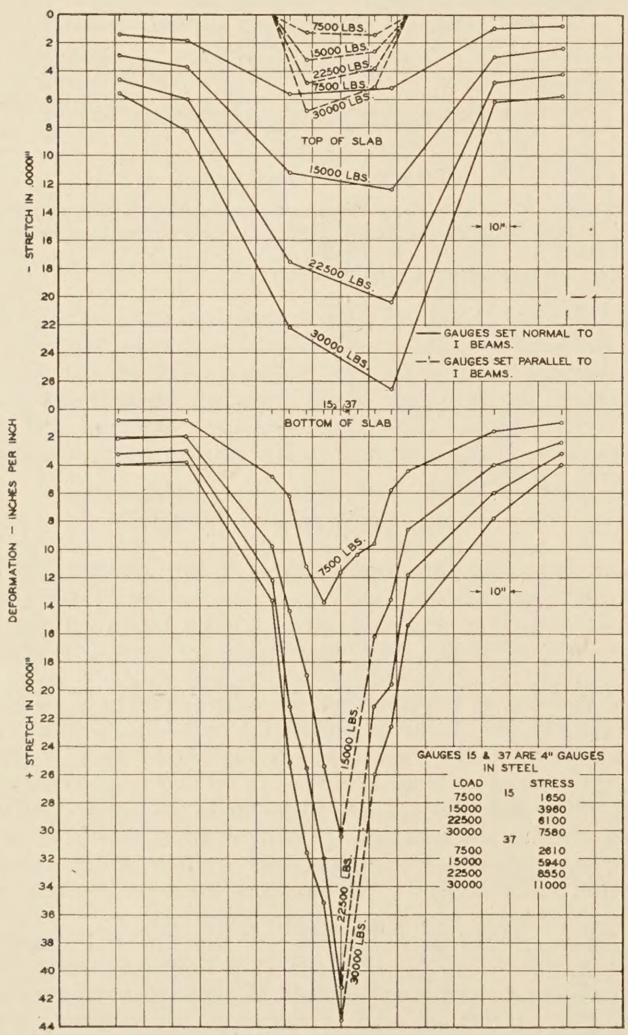


Fig. 11.—Deformations in Slab Along Axis Parallel to I Beams, Static Loads 7,500 Pounds to 30,000 Pounds

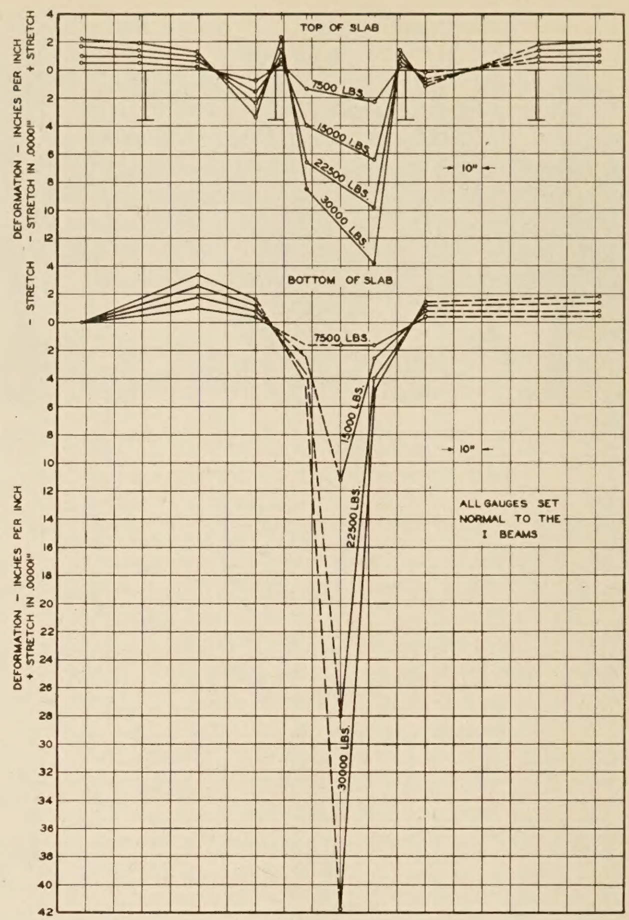


Fig. 12.—Deformations in Slab Along Axis Normal to I Beams. Static Loads 7,500 Pounds to 30,000 Pounds

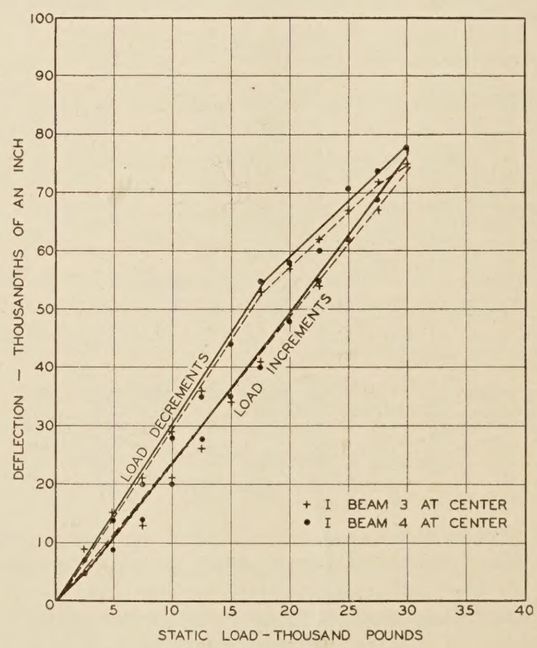


Fig. 13.—Hysteresis or Lag at Mid-span on I Beams 3 and 4 Before Impact

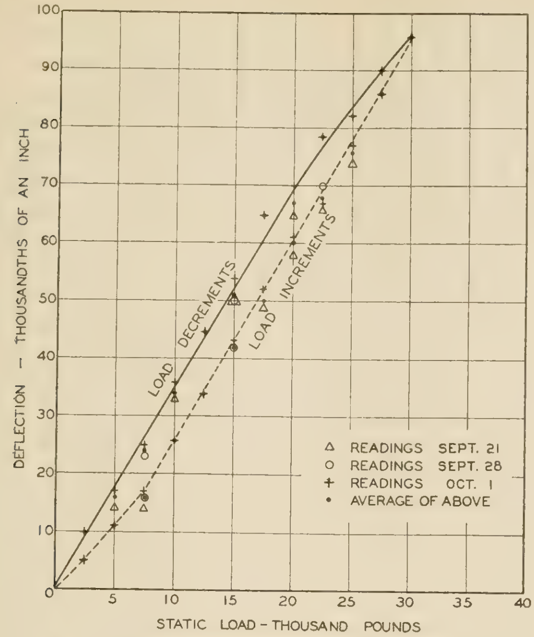


Fig. 14.—Hysteresis or Lag at Center Point of Slab Resulting from Static Loads Applied Before Impact Tests

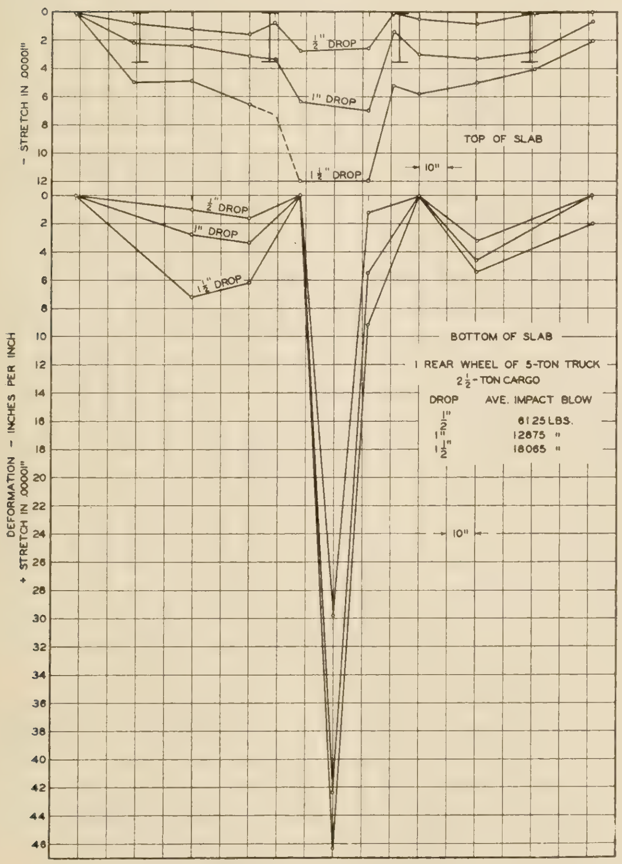


Fig. 15.—Deformations in Slab Along Axis Normal to I Beams Due to Impact Blows of 5-ton Truck with 2½-ton Load and Drops of ½, 1, and 1½ Inches

accelerometer. Deformations in both the top and the bottom of the slab were measured with graphic strain gauges. These data are shown in Table 1 and Figures 15, 16, 17, and 18.

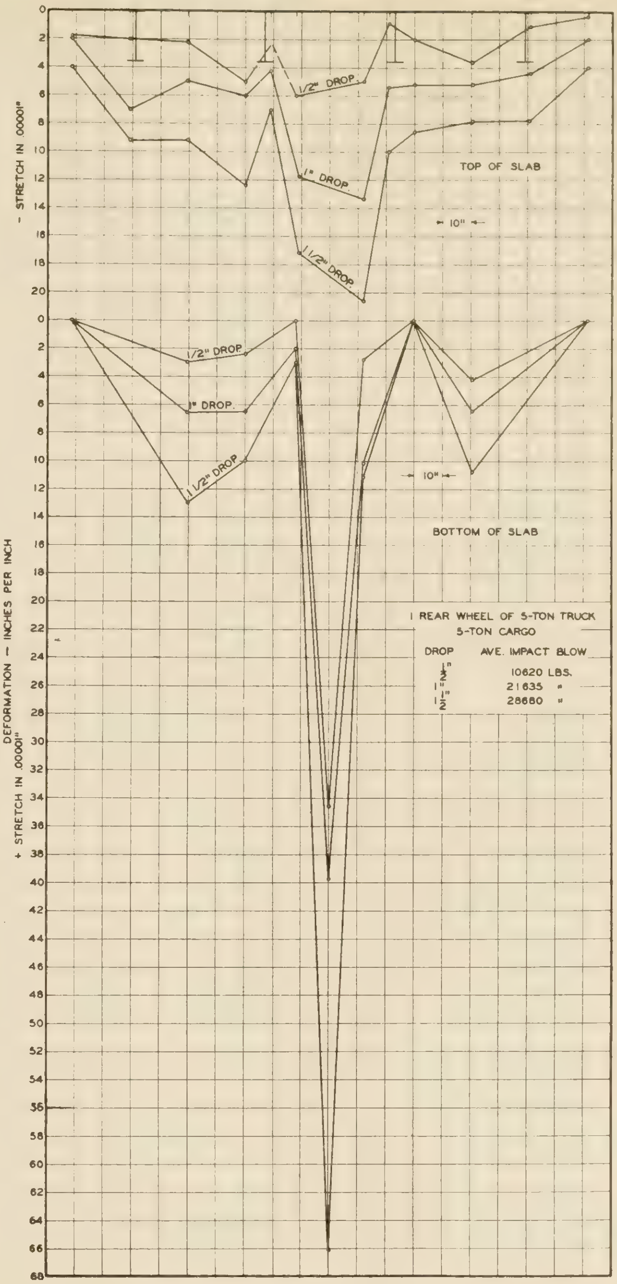


Fig. 16.—Deformations in Slab Along Axis Normal to I Beams Due to Impact Blows of 5-ton Truck with 5-ton Load and Drops of ½, 1, and 1½ Inches

The spring on the impact machine was then adjusted to simulate a 7½-ton truck with a ½-inch drop and a series of 1,000 blows was delivered. Deflection readings were taken at the beginning of the run, at the end of 250 blows and at the end of 1,000 blows as a measure of the fatigue of the slab under continued impact. The impact machine was then removed from the slab and deflections were read as a measure of the permanent set. The results of these measurements are shown in Figure 19. Deflection of the slab under these impact blows in the immediate vicinity of the point of impact was recorded by means of choked dials, and the data are shown graphically in Figure 20.

The impact machine was then moved to a quarter point of the slab centrally between **I** beams (point 6). A series of 3,000 blows was delivered at this point with a 15,000-pound wheel load and a ⅝-inch drop.

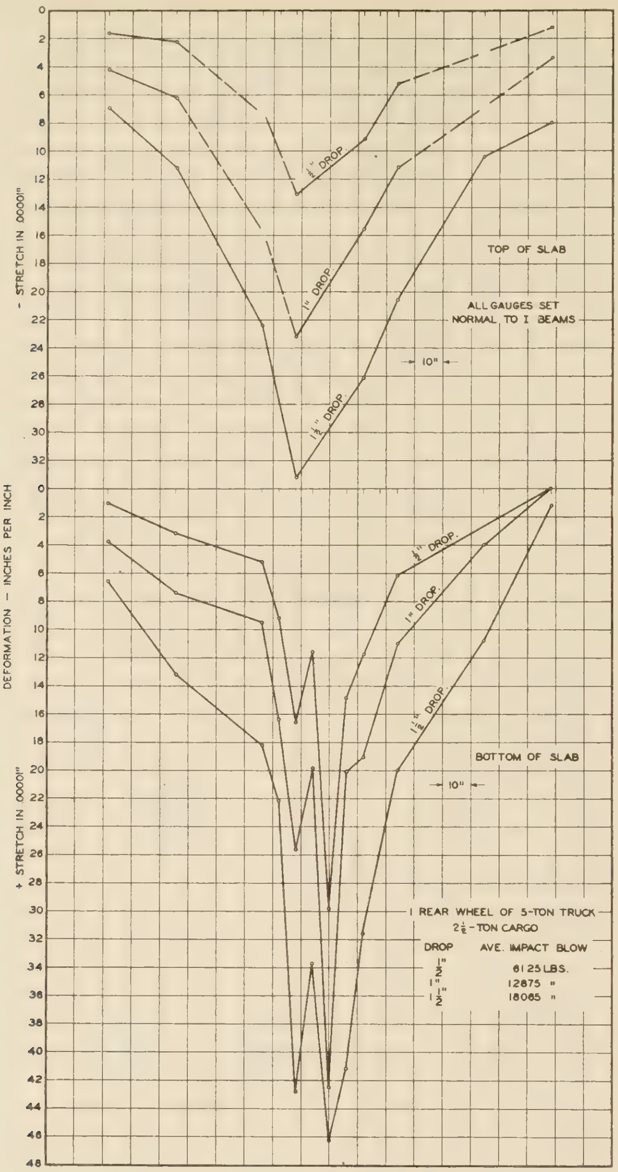


FIG. 17.—DEFORMATIONS IN SLAB ALONG AXIS PARALLEL TO I BEAMS DUE TO IMPACT BLOWS OF 5-TON TRUCK WITH 2½-TON LOAD AND DROPS OF ½, 1, AND 1½ INCHES

TABLE 1.—*Maximum stresses developed in reinforcing trusses under impact at center of slab (graphic strain gauges 15 and 37 set on bottom of two center reinforcing trusses directly below point of impact)*

Blow from one rear wheel of 5-ton truck	Drop	Stress in steel in pounds per square inch: Gauge 15	Stress in steel in pounds per square inch: Gauge 37	Impact blow	Deflection of center of slab	Date
	Inches			*Pounds*	*Inches*	
Sprung weight 3,500 pounds (2½-ton cargo); unsprung weight 1,943 pounds	½	----------	2,160	6,185	----------	Nov. 20
	1	----------	5,850	11,640	----------	Do.
	1½	----------	9,000	16,950	----------	Do.
	½	----------	2,340	6,065	0.03	Nov. 23
	1	4,370	5,850	14,110	.07	Do.
	1½	8,250	9,900	19,185	.10	Do.
Sprung weight 5,950 pounds (5-ton cargo); unsprung weight 1,943 pounds	½	----------	3,780	11,115	----------	Nov. 20
	1	----------	8,280	22,255	----------	Do.
	1½	----------	13,410	28,295	----------	Do.
	½	3,630	4,050	10,125	.05	Nov. 23
	1	8,330	9,000	21,015	.10	Do.
	1½	12,350	13,050	29,065	.15	Do.

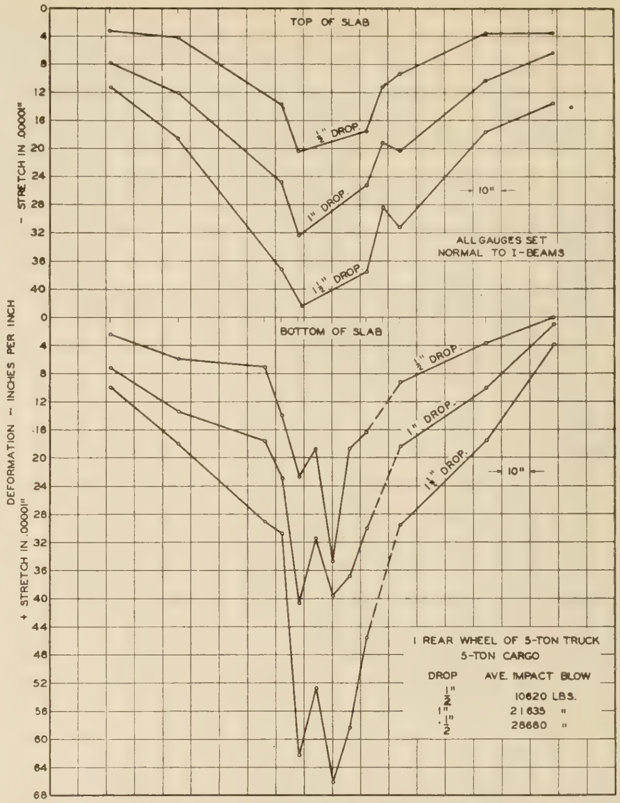


FIG. 18.—DEFORMATIONS IN SLAB ALONG AXIS PARALLEL TO I BEAMS DUE TO IMPACT BLOWS OF 5-TON TRUCK WITH 5-TON LOAD AND DROPS OF ½, 1, AND 1½ INCHES

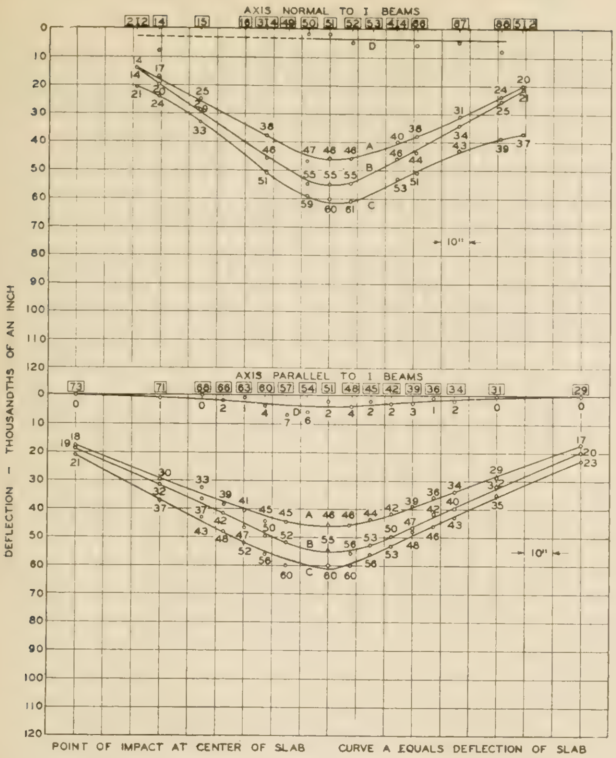
Graphic strain gauge readings were made on adjacent I-beam stringers, the gauges being 7 inches above and 7 inches below the axes of the beams and opposite the point of impact. Readings were taken at intervals of 500 blows. The data from this test are shown in Table 2.

A series of deflection readings on points 2 to 10 under impact was made at intervals of 500 blows using choked Ames dials. No marked variation in deflections was found during the series of blows. Mean deflections as found are shown in Figure 21.

The attempts made to measure any possible progressive deformation of the slab caused by repeated impact at this point were unsatisfactory. Due to the eccentric position of the point of impact there was a tendency on the part of the slab to continually shift its position on the abutments. Although this movement was small it was sufficient to cause so great a variation between the relative positions of the deflection points on the slab and floor that small changes in the slab could not be measured.

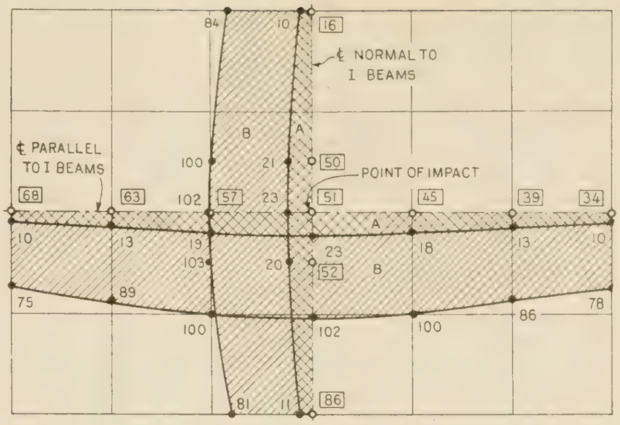
At the end of 3,000 blows very fine hair cracks had developed radially directly under the point of impact. These cracks developed for a length of about 6 inches during the first 300 blows and the remainder during the first 2,500 blows. From then on no further cracking was apparent nor did the existing cracks widen. Sketches of these cracks are shown in Figure 22.

The impact machine was then moved to quarter point 24 and a similar series of 3,000 blows with an average drop of 1¼ inches was made. In order to keep the impact machine in place it was necessary to add 2,060 pounds additional weight to the frame, thus increasing its dead weight from 14,580 pounds to 16,640 pounds. Due to the severe impact developed by this drop it



[50] Represents Deflection Points

Fig. 19.—Effect of Repeated Impact Blows on Deflection of Slab. Point of Impact at Center of Slab. Curve *A* Represents Deflection of Slab Under Weight of Impact Machine + Wheel Load (14,580+15,000 Pounds). Curve *B* Represents Deflection of Slab Under Above Load After 250 Blows of 21,080 Pounds, Wheel Load 15,000 Pounds, Unsprung Weight 2,060 Pounds with a ½-inch Drop. Curve *C* Same as Curve *B* After 1,000 Blows. On Removing Machine from Slab, Recovery was Almost Complete. Curve *D* Represents Permanent Set After Impact Machine Was Removed.



A = deflection due to wheel load.
B = total deflection under impact.
[50] = deflection points.

Fig. 20.—Deflection of Slab Due to Impact at Center of Slab. Unsprung Weight 2,060 Pounds. Wheel Load 15,000 Pounds. Average Impact Blow 21,080 Pounds. Average Drop ½ Inch. Figures Denote Mean Deflections in Thousandths of an Inch

was found necessary to guy the impact machine to the **I** beams at the corners of the slab. The entire slab, including the **I** beams, crawled diagonally on the abutments as impact blows were applied. During the first 400 blows this movement amounted to ¾ inch parallel to the **I** beams and 1 inch normal to the **I** beams.

Measurements in this series were restricted to stresses in the **I** beams adjacent to the point of impact and to deflections under impact. An increase in deflection of the slab under impact was found as the series of blows progressed. The deflection of the slab under impact is shown in Figure 23.

Fine radial hair cracks developed directly under this point of impact as at point 6. These cracks appeared for the most part during the first 600 blows and showed no increase in extent or width at the end of 3,000 blows. A sketch of these is shown in Figure 22.

The entire slab was loosened on the **I** beams and both slab and **I** beams had moved two inches at the end of the run. The slab was then jacked back to its original position on the abutment and static loads were again applied at the center in increments of 15,000 pounds up to 75,000 pounds, the safe limit of the loading device. Mean values of several check sets of deflection readings are shown in Figures 24 and 25.

Several sets of graphic strain gauge readings were taken on the two center **I** beams adjacent to the point of load with results as shown in Figure 26.

The effect of the impact tests on the load-deflection relations of the slab is shown in Figure 27.

Figure 28 shows the deflection set caused by 60,000 and 75,000 pound static loads applied after the impact tests had been completed.

Table 2.—*Stresses caused by impact in **I** beams at gauge points*

IMPACT AT POINT 24

At end of blow	4	30	100	300	400	500	600	700	1,000	1,500	2,000	2,500	3,000	Mean
	Pounds	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*
Top **I** beam 2, gauge 41	5,160	5,260	6,120	4,460	6,230	6,860	10,910	7,500	7,190	7,300	7,500	7,050	7,560	6,850
Bottom **I** beam 2, gauge 22	8,350	7,970	8,850	9,730	8,560	7,000	7,670	7,350	7,400	7,300	6,770	6,040	7,770	7,760
Top **I** beam 3, gauge 5	5,680	6,070	6,650	5,880	6,510	6,280	8,330	6,500	6,100	6,200	6,250	6,150	5,140	6,290
Bottom **I** beam 3, gauge 77	11,800	15,980	17,080	15,950	18,540	16,900	17,980	13,910	17,400	18,020	13,820	13,330	14,500	15,780

Frame of impact machine weighted with 2,060 pounds. Dead load = 14,580 + 2,060 = 16,640 pounds. Unsprung weight, 2,060 pounds. Wheel load = 15,000 pounds. Stresses read at end of impact blow. Average blow, 40,750 pounds. Average drop, 1¼ inches.

IMPACT AT POINT 6

	1	500	1,000	1,500	2,000	2,500	3,000	Mean
	Pounds	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*
Top **I** beam 2, gauge 41	1,304	2,631	2,331	1,598	2,431	2,465	3,060	2,250
Bottom **I** beam 2, gauge 22	4,850	3,293	3,782	2,880	3,707	3,734	4,880	3,875
Top **I** beam 3, gauge 5	3,284	2,970	3,020	2,385	2,650	2,570	3,683	2,940
Bottom **I** beam 3, gauge 77	2,414	3,338	3,800	3,195	2,948	3,230	5,640	3,510

Dead load = 14,580 pounds. Unsprung weight, 2,060 pounds. Wheel load, 15,000 pounds. Stresses read at end of impact blow. Average blow, 27,030 pounds. Average drop, five-eighth inch.

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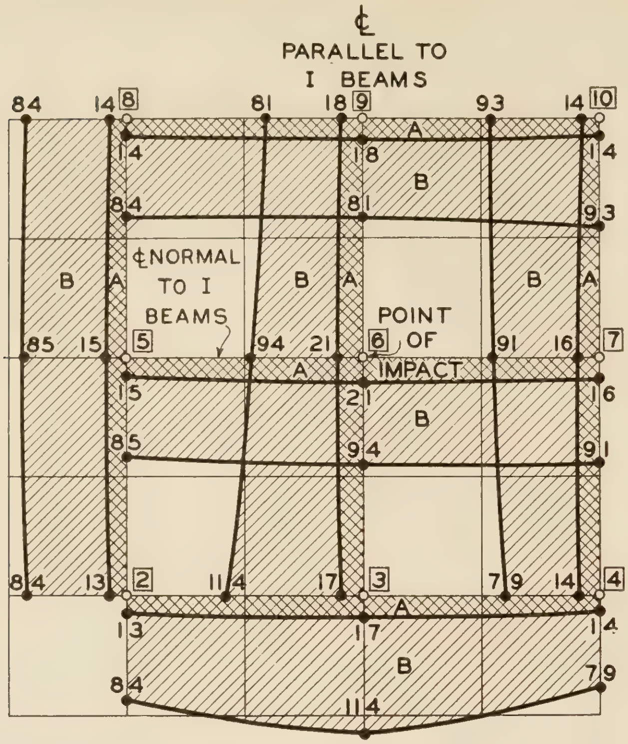


A=deflection due to wheel load
B=total deflection under impact
=deflection points.

Fig. 21.—Deflection of Slab Due to Impact at Quarter Point No. 6. Unsprung Weight 2,060 Pounds. Wheel Load 15,000 Pounds. Average Impact Blow 27,030 Pounds. Average Drop 5/8 Inch. Figures Denote Mean Deflections in Thousandths of an Inch.

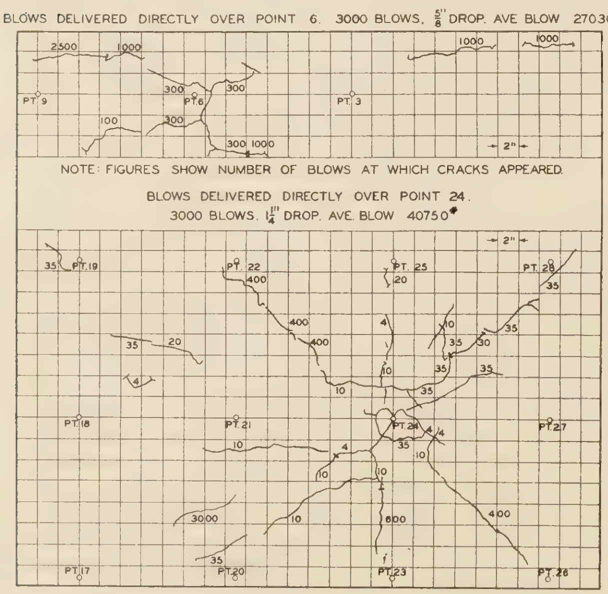
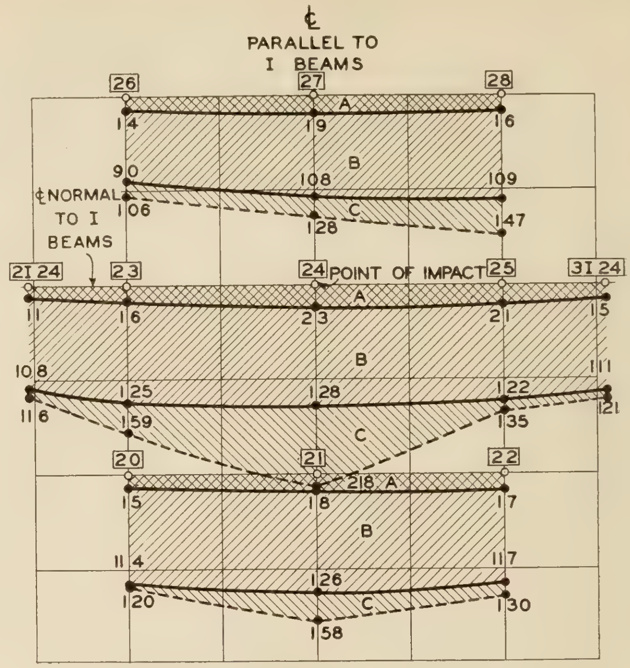


Fig. 22.—Sketches Showing Crack Development as a Result of Impact Blows on Point 6 and Point 24

Check measurements of the spread or tilting of the I beams under 15,000-pound load increments were made as described on page 161. Figure 29 shows that, in spite of the fact that the slab had been loosened from the I beams, the I beams spread or tilted enough to allow their flanges to follow the deflection of the slab.



A=deflection due to wheel load.
B=total deflection under impact at beginning of run.
C=increase in total deflection under impact at end 3,000 blows
=deflection point

Fig. 23.—Deflection of Slab Due to Impact at Quarter Point 24. Unsprung Weight 2,060 Pounds. Wheel Load 15,000 Pounds. Average Impact Blow 40,750 Pounds. Average Drop 1¼ Inches. Figures Denote Mean Deflections in Thousandths of an Inch.

Fine hair cracks as shown in Figure 30 developed under static loads of 60,000 to 75,000 pounds. These cracks were very fine and showed no increase in extent or width under repeated loading.

On the removal of the slab one of the reinforcing trusses was broken out of the slab and it was found that in spite of the dry mix used, about 90 per cent of the spaces in the upper chord channels were filled with concrete.

TESTS ON SECOND SLAB

The materials used in the construction of the second slab were identical with those used in the first slab. The cement showed a tensile strength of 300 pounds per square inch and 415 pounds per square inch at the end of 7 and 28 days, respectively. Concrete test cylinders developed a modulus of elasticity of 2,883,000 pounds per square inch and a compressive strength of 3,560 pounds per square inch at the end of 28 days.

The methods of placing and curing were the same in both slabs. The mix was the same, except that the water content was increased, an average slump of 8.7 inches with a 4 by 8 by 12 inch cone, and an average flow of 154 being maintained.

In the tests on the second slab the following measurements were made:

a. Deflection of the slab and of the **I** beams under static loads.
b. Deformations in the slab and in the **I** beams under static loads.
c. Spread of two center **I** beams under static loads.
d. Reactions at one end of the **I** beams.

In testing the second slab measurements were to a great extent confined to one quadrant as it was found

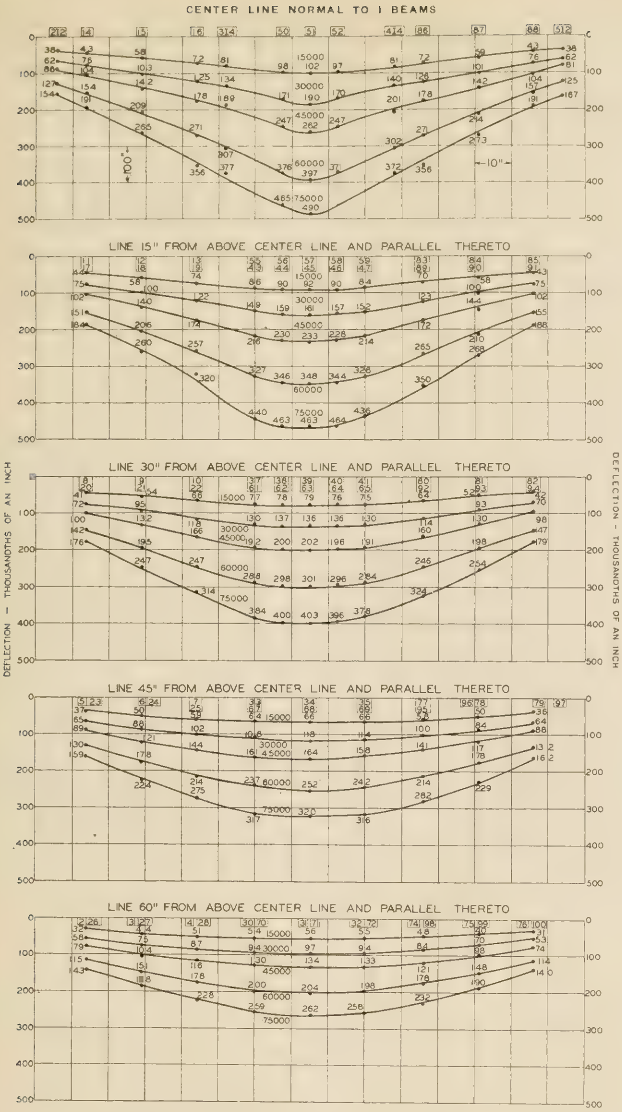


FIG. 24.—SLAB DEFLECTION UNDER INCREMENTS OF STATIC LOADING UP TO 75,000 POUNDS. LOADING APPLIED AFTER IMPACT TESTS

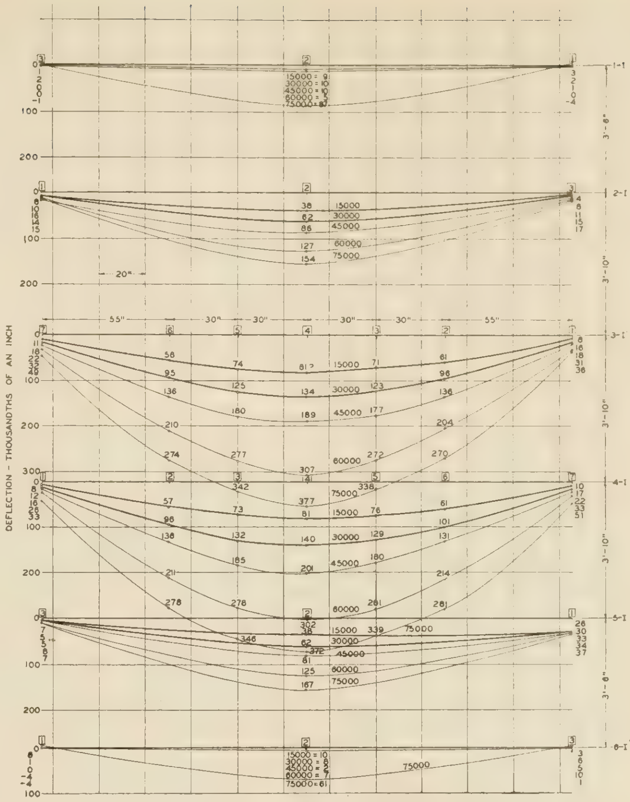


FIG. 25.—DEFLECTION OF I BEAMS UNDER INCREMENTS OF STATIC LOADING UP TO 75,000 POUNDS. LOADING APPLIED AFTER IMPACT TEST

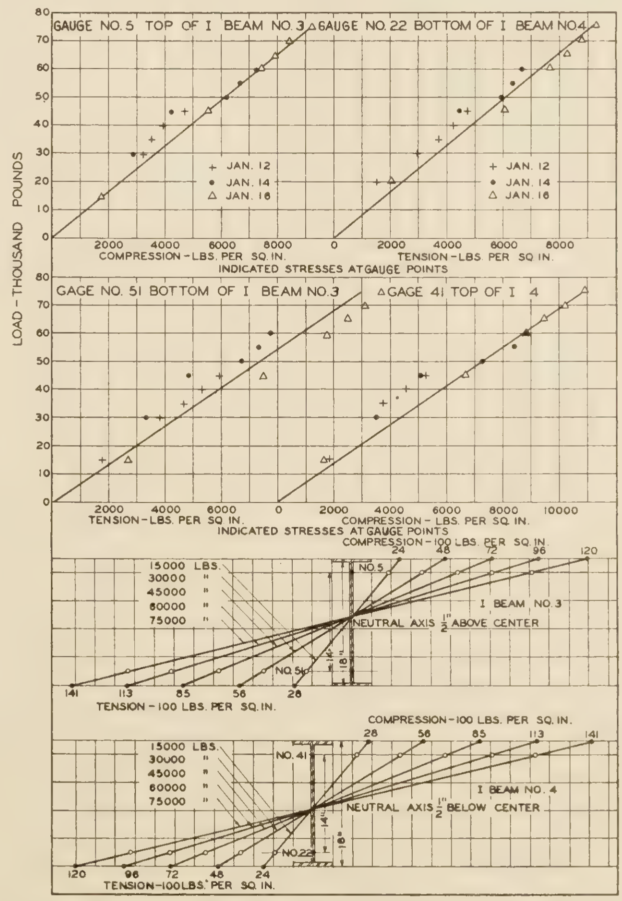


FIG. 26.—POSITION OF NEUTRAL AXIS AT CENTER OF TWO CENTER I BEAMS UNDER STATIC LOAD APPLIED AT CENTER OF SLAB

from the first slab that all four quadrants behaved symmetrically.

Static loads were applied at the same point and in the same manner as in the first slab, except that a circular steel plate 12 inches in diameter and 5/8 inch thick and a cushion of 3/8-inch rubber packing were substituted for the cast-iron block and tire sections used on the first slab, thus giving a more definite area of load.

Deflections were measured by wooden rods fitted with metal shoes on the lower end and Ames dials at the top set permanently between the gauge points on the slab and on the **I** beams and those on the floor. (Fig. 31.) The layout of deflection points and graphic strain gauges is given in Figure 32.

Deformations in the slab and stresses in the **I** beams were determined by graphic gauges attached as in the previous slab. No stresses were measured in the reinforcing steel in the second slab.

The spread of the two center **I** beams was measured by the same method as in the first slab, except that two Ames dials were used on each staff, one at either end.

The apparatus used in the measurement of the end reactions of the **I**-beam stringers consisted of a 2 by

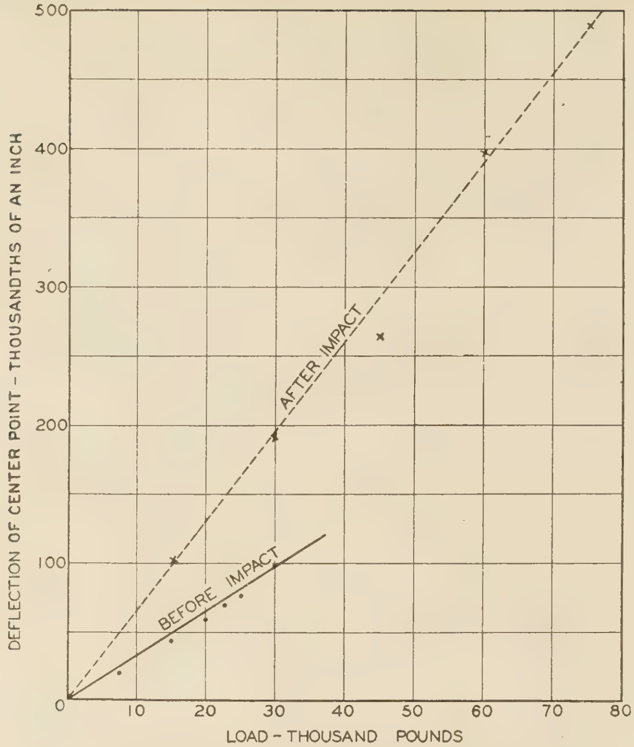


FIG. 27.—DIAGRAM SHOWING LOAD-DEFLECTION RELATION FOR STATIC LOADS APPLIED BEFORE AND AFTER IMPACT TESTS

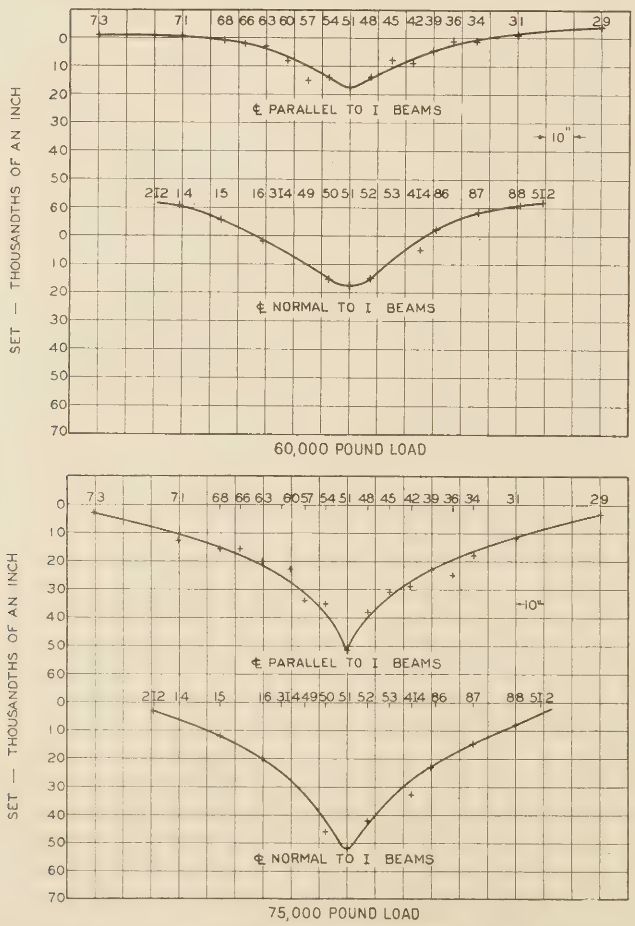


FIG. 28.—DIAGRAMS SHOWING SET CAUSED BY 60,000 AND 75,000 POUND STATIC LOADS APPLIED AFTER IMPACT

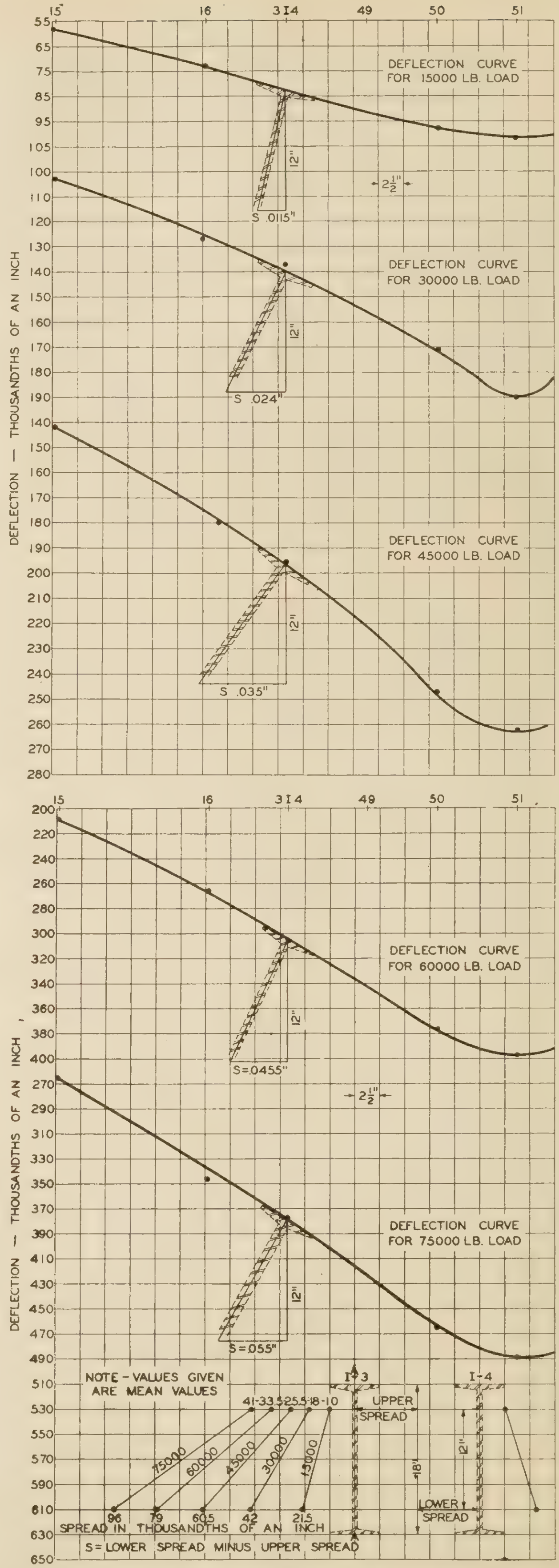


FIG. 29.—SKETCHES SHOWING SPREAD OF THE TWO CENTER I BEAMS UNDER VARIOUS STATIC LOADS

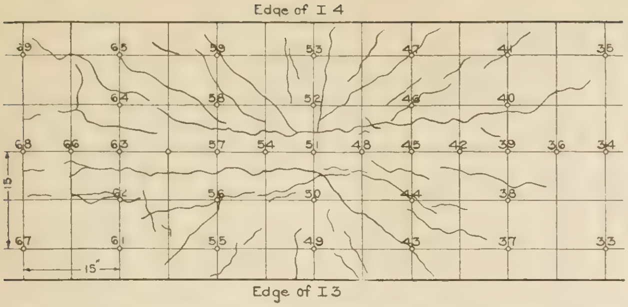


Fig. 30.—Showing Cracks which Became Visible Under Loading Point Upon Application of Static Loads From 60,000 to 75,000 Pounds

4 inch steel beam, 20 feet long, rigidly supported on edge directly above the ends of the **I** beams by struts resting on the concrete abutment. (Fig. 4.) Six suspension rods of ⅝-inch round manganese steel were attached to this beam at their upper ends and to the **I** beams by suitable stirrups at their lower ends. Nuts at the upper end of these suspension rods permitted them to be shortened, thus takiug up the load of the **I** beams. On opposite sides of the suspension rods gauge points on 15-inch centers permitted the elongation of the rods under load to be measured with a 15-inch Berry strain gauge. Each rod was calibrated and from the mean elongation of the two sets of gauge points the load on each **I** beam could be determined.

Fig. 31.—Wooden Rods in Place for the Measurement of Deflections of Second Test Slab

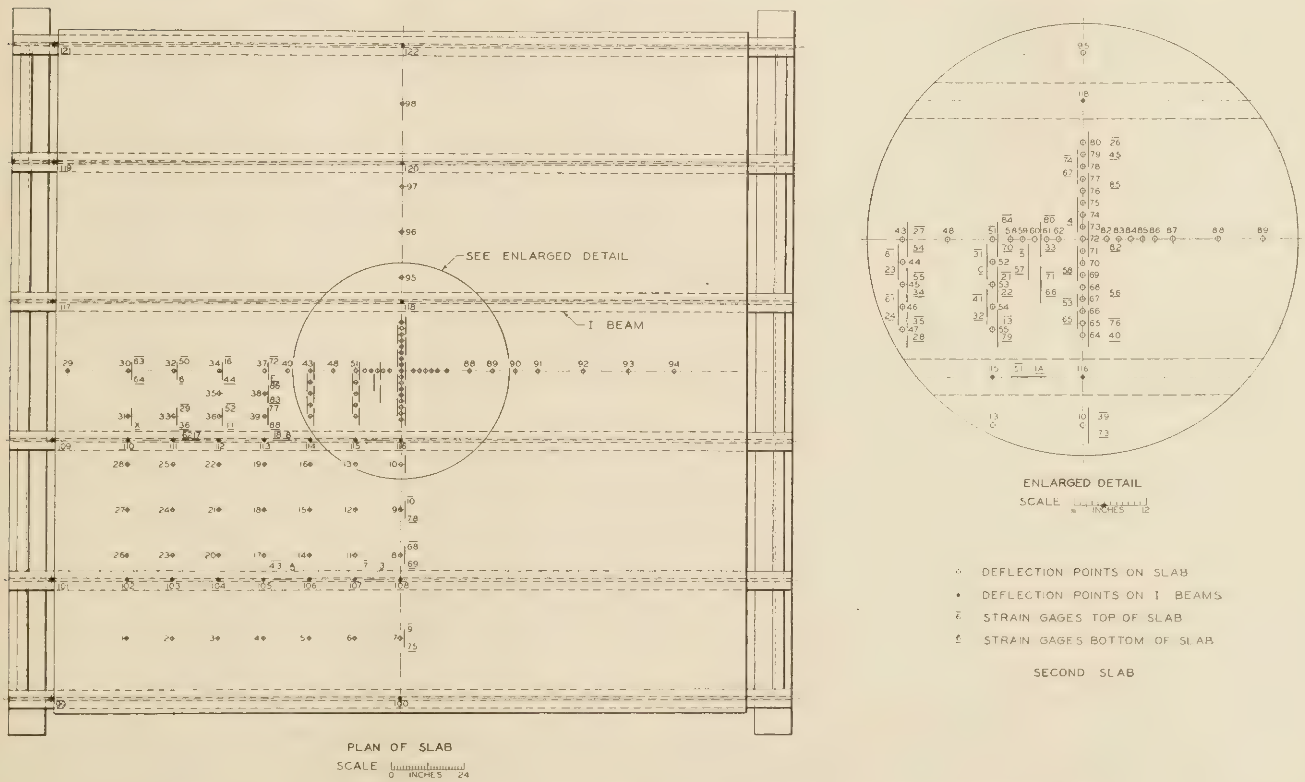


Fig. 32.—Layout of Deflection Points and Graphic Strain Gauges for Second Slab

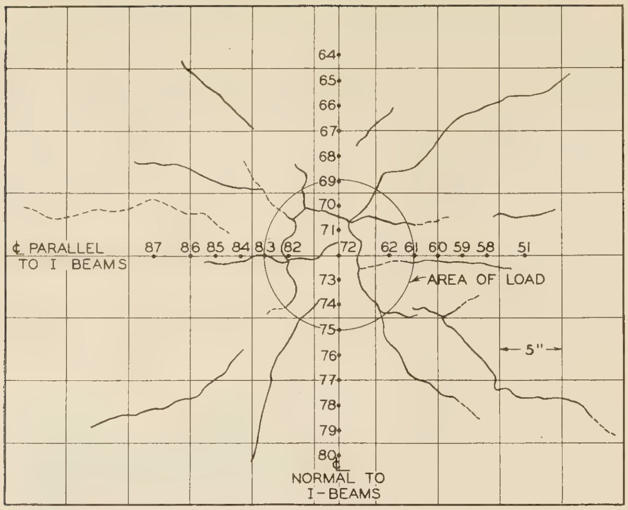


Fig. 33.—Sketch Showing Cracks Developed by Static Loads at Center Point. Solid Lines Show Hair-line Cracks Developed by Loads from 31,250 to 40,000 Pounds. Dash Lines Show Increase Due to 75,000-pound Load

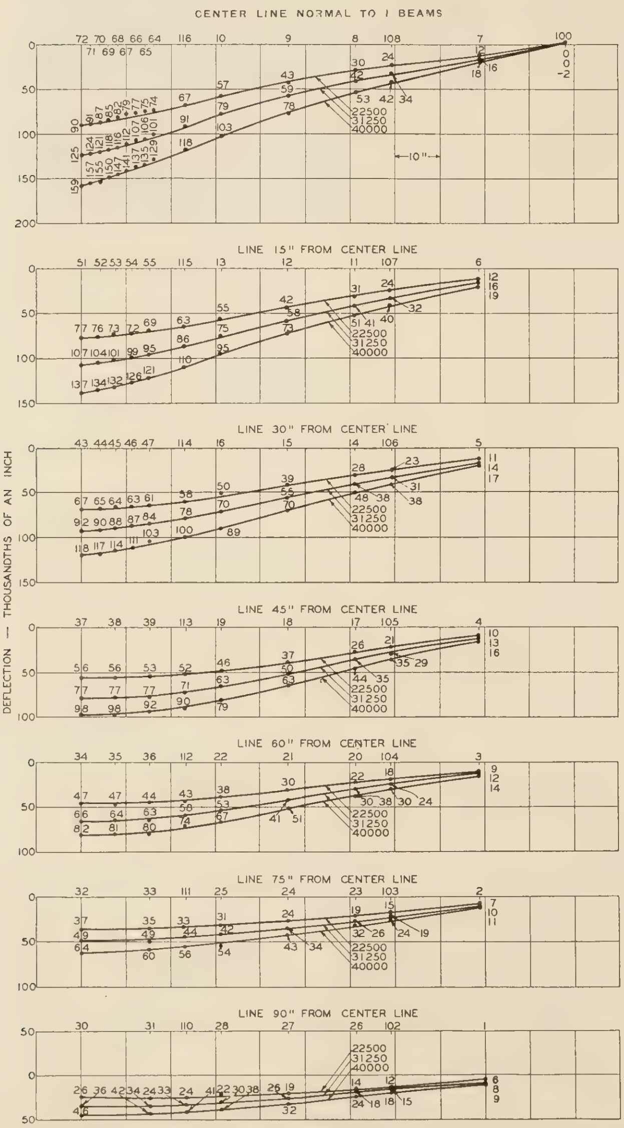


Fig. 34.—Deflection of Second Slab under Different Static Loads Applied at the Center

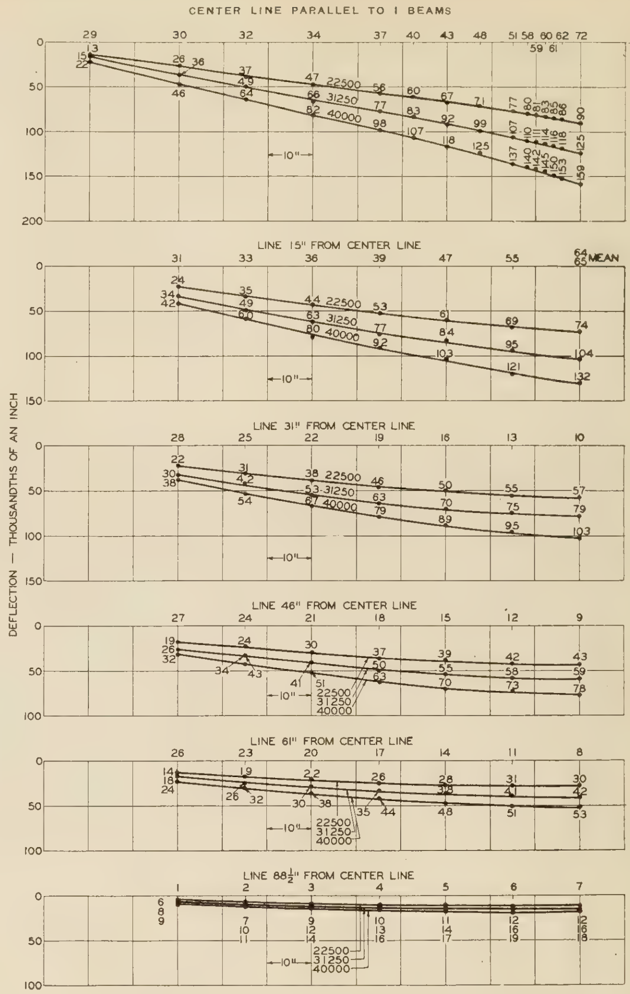


Fig. 35.—Deflections of Second Slab under Different Static Loads Applied at the Center

The procedure was as follows:

Wooden rods with Ames dials at their upper ends were set under the deflection points at the ends of I beams nearest the suspension rods. These deflection dials were set to read zero. Several sets of initial readings were taken on each suspension rod with these rods loose. The suspension rods were then shortened sufficiently to allow the bearing plates to be slipped from beneath the I beams and the rods were then adjusted to bring the Ames dials to zero or their original position with bearing plates in place. Strain gauge readings were now taken on each suspension rod as a measure of the dead load of the slab. Static loads were then applied and each time the dials under the ends of the I beams were adjusted by means of the suspension rods to show the same deflection of the I beam that appeared under static load with bearing plates in place. Strain gauge readings were then taken as a measure of the reactions produced by static loads.

PROGRAM FOLLOWED FOR SECOND SLAB

Five check sets of deflection readings were taken for static loads, applied at the center of the slab, of 22,500, 31,250, and 40,000 pounds. Radial cracks as shown in Figure 33 developed under the two larger loads. The mean values of the deflections as measured are shown in Figures 34 and 35.

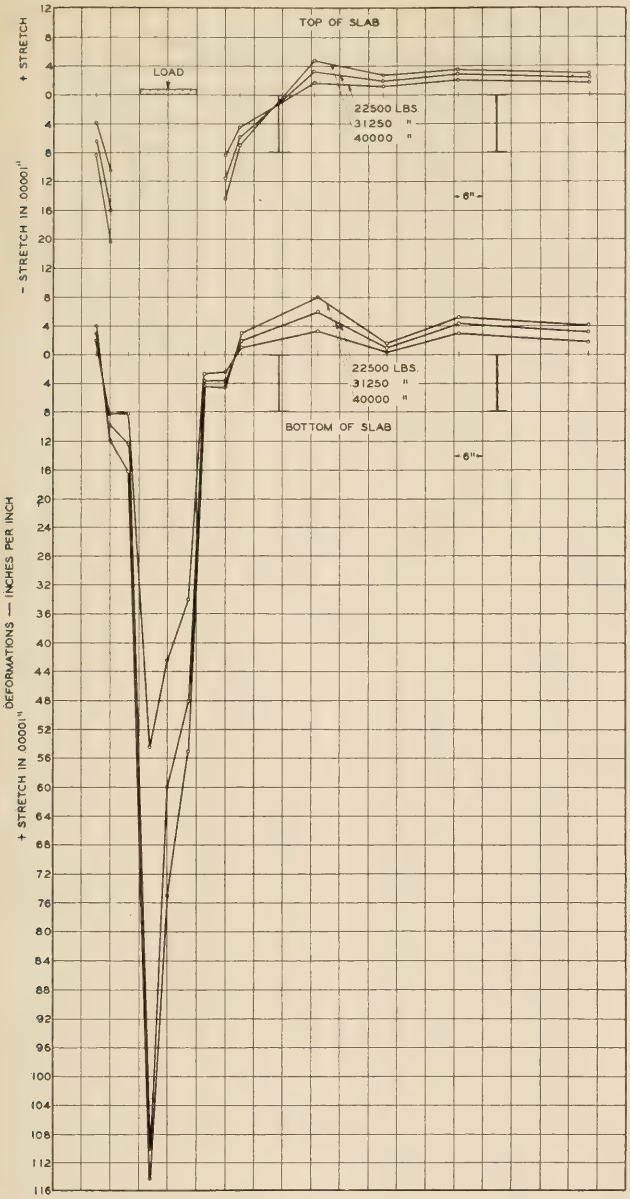


Fig. 36.—Deformations in Slab along Axis Normal to I Beams under Static Loads of 22,500, 31,250, and 40,000 Pounds

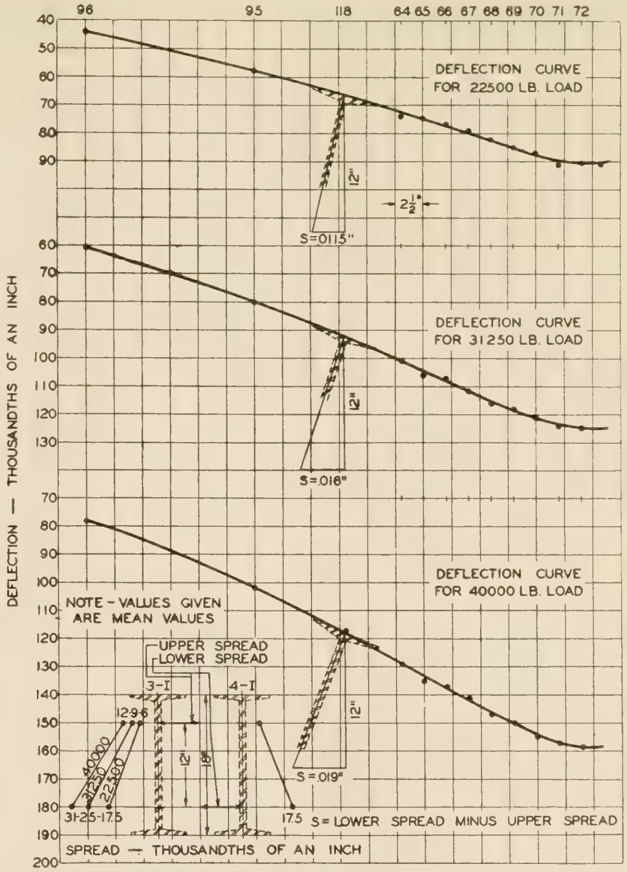


Fig. 37.—Sketch Showing Spread of Two Center I Beams under Static Load during Test of Second Slab

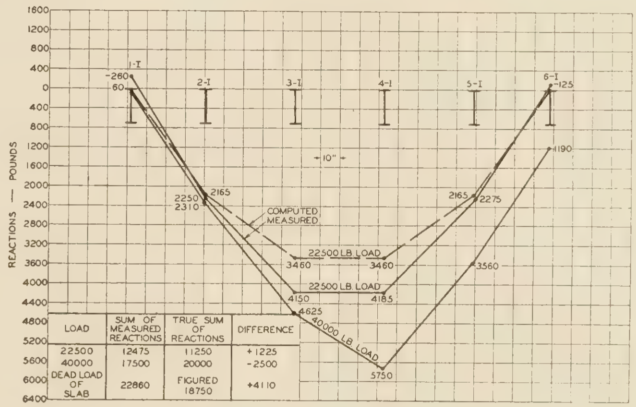


Fig. 38.—I-beam Reactions as Measured for Loads of 22,500 and 40,000 Pounds. Reactions for the 22,500-pound Loading Are the Mean of Five Sets of Readings and for the 40,000-pound Loading from One Set. The Computed Reactions are Derived from Deflections of I Beams under Load While Resting on Shims

Check sets of graphic strain gauge measurements were made in the top and bottom of the slab. As in all of the strain gauge measurements on this slab the recorded deformations were erratic. It is thought that this is due to local stresses rather than to any fault in the gauges or the technic. When the second slab was built it was necessary to remove bows in the reinforcing trusses by wiring the trusses to the forms. After the concrete set up, this distortion of the trusses must have produced some stress in the concrete which would tend to disturb the measured deformations, particularly under the smaller loads. Deformations are shown in Figure 36.

Six sets of measurements of the spread of the **I** beams were made under the above static loads with results as shown in Figure 37.

Five sets of reaction measurements at one end of the **I** beams were then made for each of the above loads. Reactions were also measured for loads from 5,000 pounds to 30,000 pounds by increments of 5,000 pounds. Results of reaction measurements are shown in Table 3 and Figures 38, 39, and 40.

Check sets of slab deflections were then measured for static loads of 30,000 to 75,000 pounds in 15,000-pound increments with results as shown in Figure 41. Under the 75,000 pound load cracks as developed by the 40,000-pound load were slightly increased, as shown in Figure 33. These cracks were hair cracks and of no serious importance.

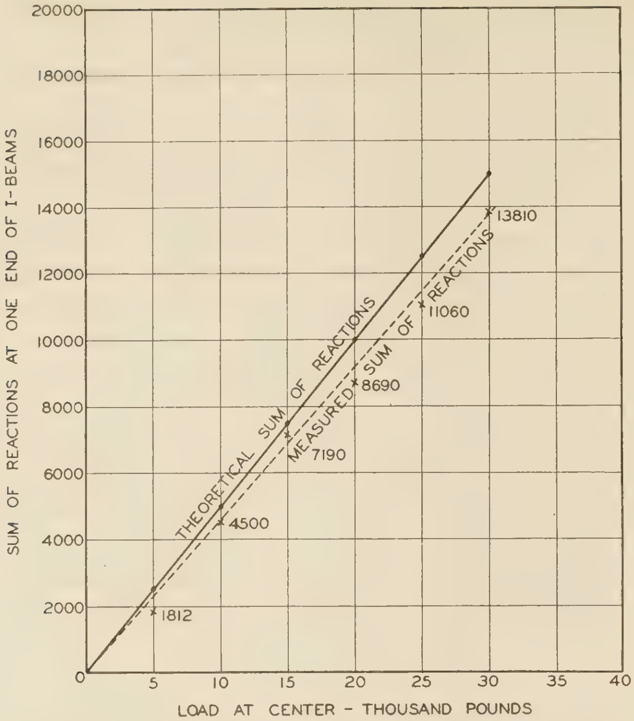


FIG. 39.—VARIATION OF SUM OF MEASURED REACTIONS FROM THEORETICAL VALUES

TABLE 3.—*Reactions as measured at I beams for 5,000-pound load increments*

[Computed dead load, 37,500 pounds]

Loading	I beam No. 1	2	3	4	5	6	Total	Difference between true and measured reactions
Pounds	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*
Slab only	2,560	4,560	2,500	5,690	125	4,625	20,060	------------
5,000	−310	440	440	875	685	−310	1,820	−680
10,000	−250	1,125	1,060	1,690	1,000	−125	4,500	−500
15,000	−250	1,750	1,560	2,810	1,500	−190	7,190	−310
20,000	−310	1,690	2,560	3,440	1,875	−565	8,690	−1,310
25,000	440	2,125	2,875	4,435	2,440	−375	11,060	−1,440
30,000	−250	2,250	4,190	5,125	2,810	−310	13,810	−1,190

RESULTS INDICATE THAT SLABS ARE ADEQUATE FOR DESIGN LOAD

Regarding these tests purely as a measure of the ability of the floor system to perform safely the duties imposed on it, the following facts are evident.

The design load assumed for the floor slabs was a 15,000-pound static load plus a 50 per cent allowance for impact, or a total load of 22,500 pounds. The first slab under a static load of 30,000 pounds, which is one-third greater than the design load, showed no signs of distress or permanent set.

Under loads of 22,500 pounds and 30,000 pounds the maximum indicated stresses in the reinforcing steel of the first slab directly under the point of load were 8,550 and 11,000 pounds per square inch, respectively, based on the measured value of E of 30,000,000 pounds per square inch.

Referring to Figure 26, it will be seen that for a static load of 30,000 pounds the maximum measured

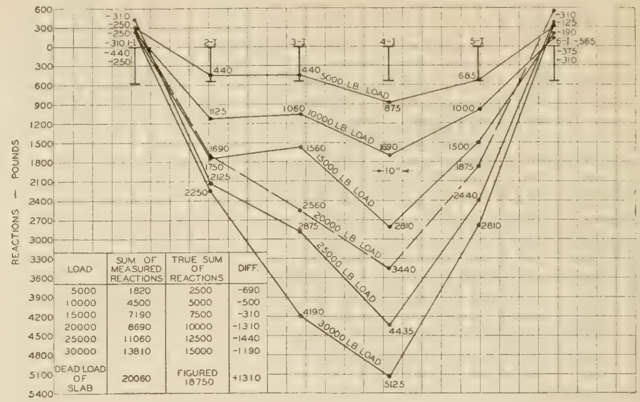


FIG 40.—I BEAM REACTIONS AS MEASURED FOR 5,000-POUND LOAD INCREMENTS FROM 5,000 TO 30,000 POUNDS

deformations in the I beams indicate a stress of 4,500 pounds per square inch in the steel, at the gauge points, based on a measured value of $E=30,000,000$ pounds per square inch. If a straight line variation of stress across the I beam is assumed, the indicated maximum fiber stress is 5,600 pounds per square inch at the extreme fiber.

Four sets of strain gauge readings in the concrete on the upper surface of the slab, one typical set of which is shown in Figures 11 and 12, indicate that the 22,500 and 30,000 pound loads caused maximum compressive stresses of 470 and 580 pounds per square inch, respectively, along the axis normal to the I beams, while along the axis parallel to the I beams maximum compressive stresses of 930 and 1,210 pounds per square inch were created by the same two loads. These are the maximum stresses indicated in any of the four sets of measurements and not the maxima indicated by the single set shown in Figures 11 and 12

Slight negative bending moments are indicated in the top of the slab directly over the I beams and along the center line of the slab normal to them, and if it can be assumed that no cracking occurred in the concrete, the deformations measured would indicate tensile stresses in the concrete of 130 pounds per square inch and 160 pounds per square inch for the 22,500-pound and 30,000-pound loads, respectively.

Although deformations equivalent to maximum tensile stresses in the concrete of 1,900 and 2,400-pounds per square inch were found in the bottom of the slab at the center under the above static loading the reinforcing steel was ample and no visible cracks developed. The loadings referred to were static.

Fatigue due to the dead load of the impact machine of 14,580 pounds plus repeated impact blows of about 21,000 pounds was apparent but recovery was almost complete after an interval of rest.

Under an impact blow of about 29,000 pounds a maximum stress of somewhat over 13,000 pounds per square inch was developed in the reinforcing trusses directly under the blow. For the same blow a maximum compression in the slab of 855 pounds per square inch along the axis normal to the I beams and 1,764 pounds per square inch along the axis parallel to the I beams was found.

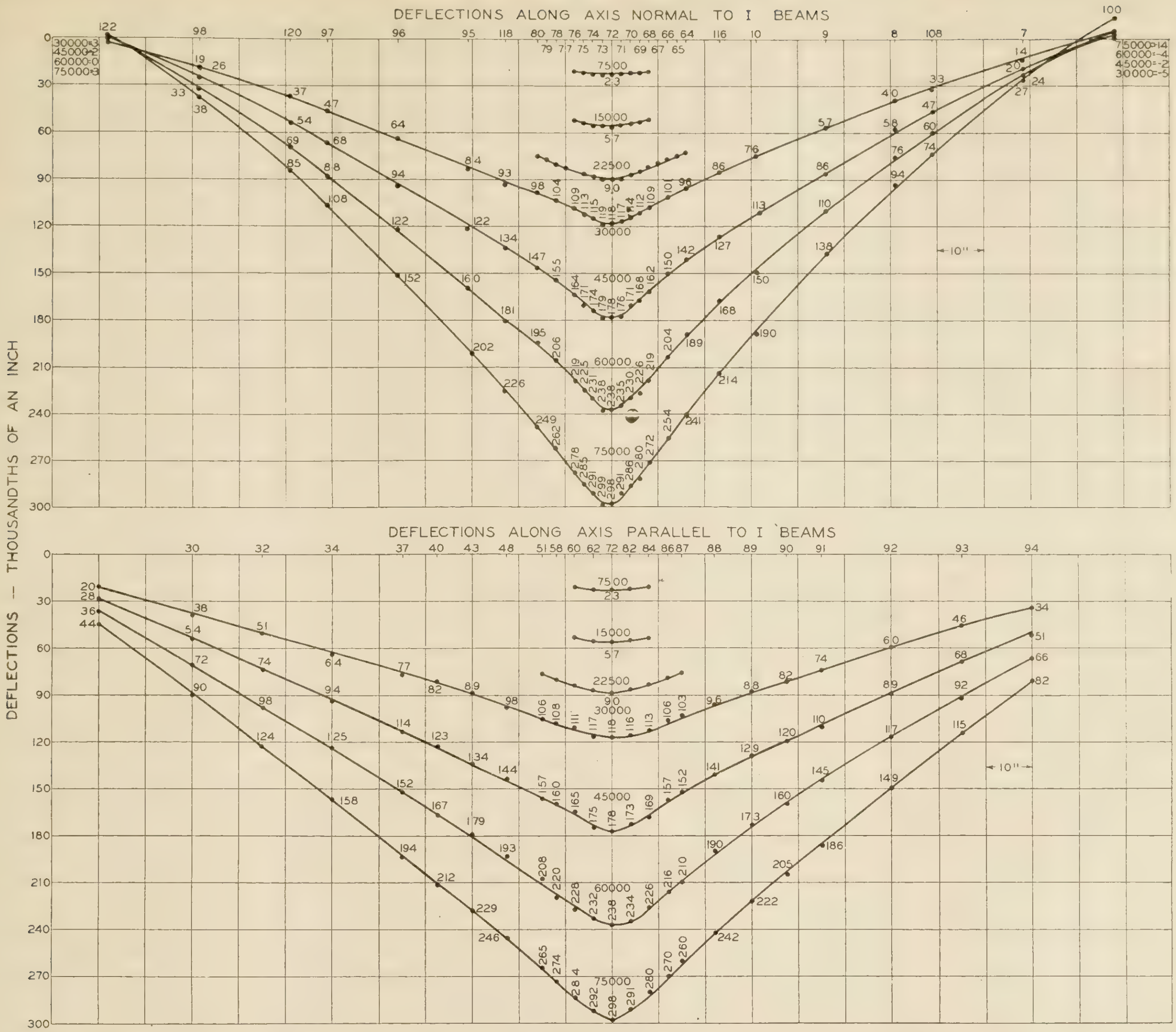


Fig. 41.—Deflection of Second Slab Under Static Loadings from 30,000 to 75,000 Pounds. Values Used Are Mean Deflections

Deformations equivalent to a maximum tensile stress in the concrete of 2,754 pounds per square inch were found in the bottom of the slab under the 29,000-pound impact pressure, yet no visible cracks appeared.

A series of 3,000 impact blows of about 27,000 pounds intensity at a quarter point on the slab developed a maximum mean indicated stress at the gauge points of 3,875 pounds per square inch in the **I** beams, while a series of 3,000 blows of about 41,000 pounds intensity at a second quarter point resulted in a mean indicated stress at the gauge points, of 15,780 pounds per square inch in the **I** beams. (See Table 2.)

Some few hair cracks developed directly under the point of impact at both quarter points, but as these cracks developed during the early part of each series of blows, were not progressive, and showed practically no increase at the end of the run, they are not to be considered as of serious importance. Although the first slab showed a marked increase in flexibility due to continued impact at the center no signs of distress were apparent until a static load of 60,000 pounds had had been applied. Under loads from 60,000 to 75,000 pounds, hair cracks similar in type to those at the quarter points developed on a limited area directly under the point of load. It is probable that the increase in flexibility is due, in part to fatigue, and in part to a decrease in the T-action of the slab and **I** beams, caused by a breaking of the bond between the slab and the **I** beams by the series of severe impact blows.

The increased water content used in the second slab caused a decrease in compressive strength of 35 per cent (from 5,465 pounds to 3,560 pounds per square inch) and under the 30,000 pound load an increase in deflections or flexibility of about 18 per cent with a corresponding increase in deformations and stresses. This slab under a static load of 31,250 pounds evidenced no signs of distress. At some point between loads of 31,250 pounds and 40,000 pounds a few fine hair cracks developed directly under the load, but an increase of load to 75,000 pounds caused very slight increases in these cracks, which may be taken as evidence that the load was well taken care of by the reinforcing steel. As the concrete actually used on the bridge floor slabs showed a mean compressive strength of 2,700 pounds per square inch probably due to its greater water content, increased deflections and stresses over those found in

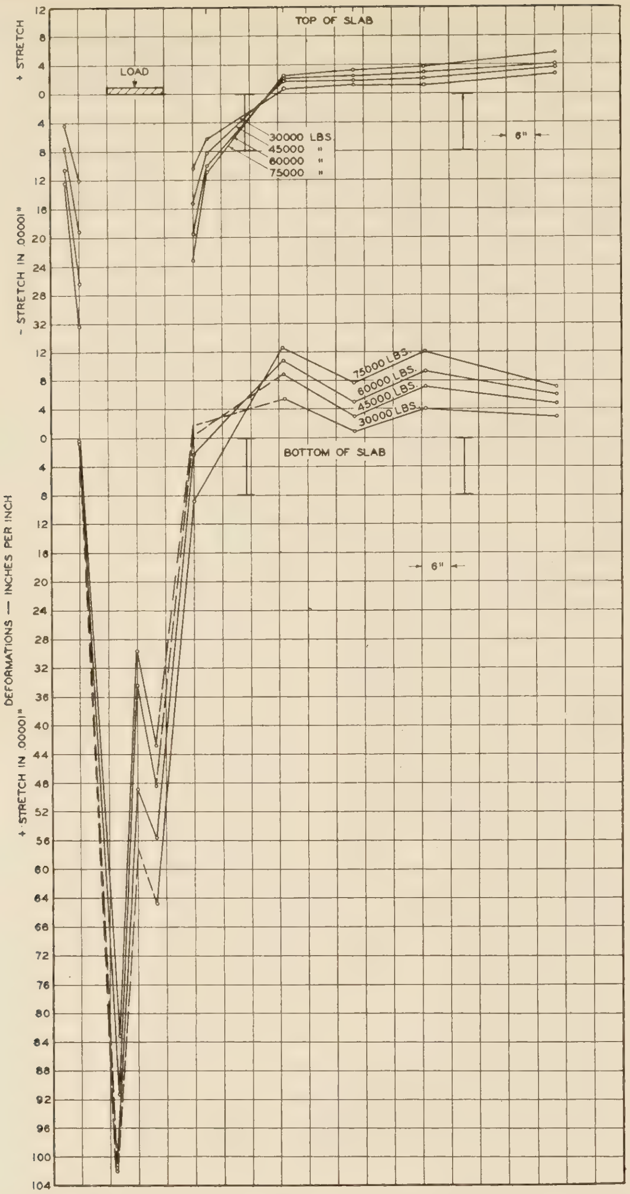


Fig. 42.—Mean Deformations in Slab Along Axis Normal to I Beams for Static Loads of 30,000 Pounds to 75,000 Pounds

either of the test slabs are to be expected, but as the factors of safety are large enough to care for such an increase it is quite apparent that the slabs are ample to withstand any possible loading to which they may be subjected by modern traffic.

RESULTS CONSIDERED AS A POSSIBLE AID TO DESIGN

As the data collected during these tests will undoubtedly be of value to those who are interested in developing a method of design, they are presented as fully as possible in this report, with a discussion of their relative value and probable accuracy.

It is felt that of the measurements made in these tests, the deflection measurements are the most reliable because:

1. The mechanical methods used in making them allow but small chance for physical errors to creep in.
2. Repeated sets of readings on both slabs give close checks for the individual slabs.
3. Deflection readings for both the first and second slab check each other closely as to type of deflection curve.

It will be seen from Figures 8, 24, 34, 35, and 41 that both slabs deflect symmetrically about their axes and that deflections vary directly with the static loads.

From Figures 9 and 25 it appears that the **I**-beam stringers also deflect symmetrically, and with the slab, as deflections of the **I** beams coincide with the deflection curves of the slabs. Figures 29 and 37 clearly show that the flanges of the **I** beams tilt and follow the curve of the slab, the entire slab deflecting as a simple plate being slightly stiffer along the axis normal to the **I** beams. The above action is quite evident from Figure 43.

The first slab under static loads up to 30,000 pounds showed no permanent set, although the expected lag was observed in both slab and steel, as shown in Figures 13 and 14.

Progressive fatigue with increased flexibility under continued impact blows delivered at short intervals with almost complete recovery after an interval of rest is evident from Figure 19. From Figures 8, 24, and 27 it will be seen that deflections of the slab under equal static loads have been almost doubled by the fatigue caused by 7,000 impact blows and decrease in **T**-beam action of the **I** beams and slab acting as a unit.

Figures 8 and 24 indicate that the type of deflection curve after impact is the same as that before impact, varying only in amount. This is more clearly shown in Figure 44, where deflection curves for the first slab both before and after impact and for the second slab are reduced to type curves. Deflection curves for the first slab under a static load of 30,000 pounds before impact were taken as base curves. Curves for the first slab under static load after impact and the second slab under static load were reduced thereto by applying factors equal to the ratios of the deflections at the center points under 30,000 pound loads to the center deflections of the base curve.

It appears from these type curves that the above three cases do not vary in type but vary only in the amount of the deflections, due probably to fatigue and increased flexibility in the case of the first slab after impact and to the weaker concrete in the second slab. Therefore it may be assumed that the action of the slabs in all three cases was identical.

In Figure 45 a comparison of the deflections of the first slab under static load both before and after impact with the deflections as measured by "choked" dials under an impact blow equal to the static load is shown. This comparison shows the progressive fatigue due to the impact and also indicates that the type of the deflection curves is essentially the same for both static loads and impact blows of equal magnitude.

In Figure 46 a comparison is made of the stresses in the reinforcing trusses under impact and static pressures, by drawing the curve which shows the static load-stress relation (data from Figure 10) and then plotting the impact data shown in Table 1 against this curve. The indication is that somewhat higher stresses result from impact pressures than do from static pressures of the same magnitude. The stresses under static load appear to vary directly with the load. This is also true of the static load stresses in the **I** beams as shown in Figure 26.

Stresses as measured in the concrete of the two slabs were more or less erratic and unsatisfactory. The reinforcing trusses on account of their weight, length, and method of fabrication were warped in both planes,

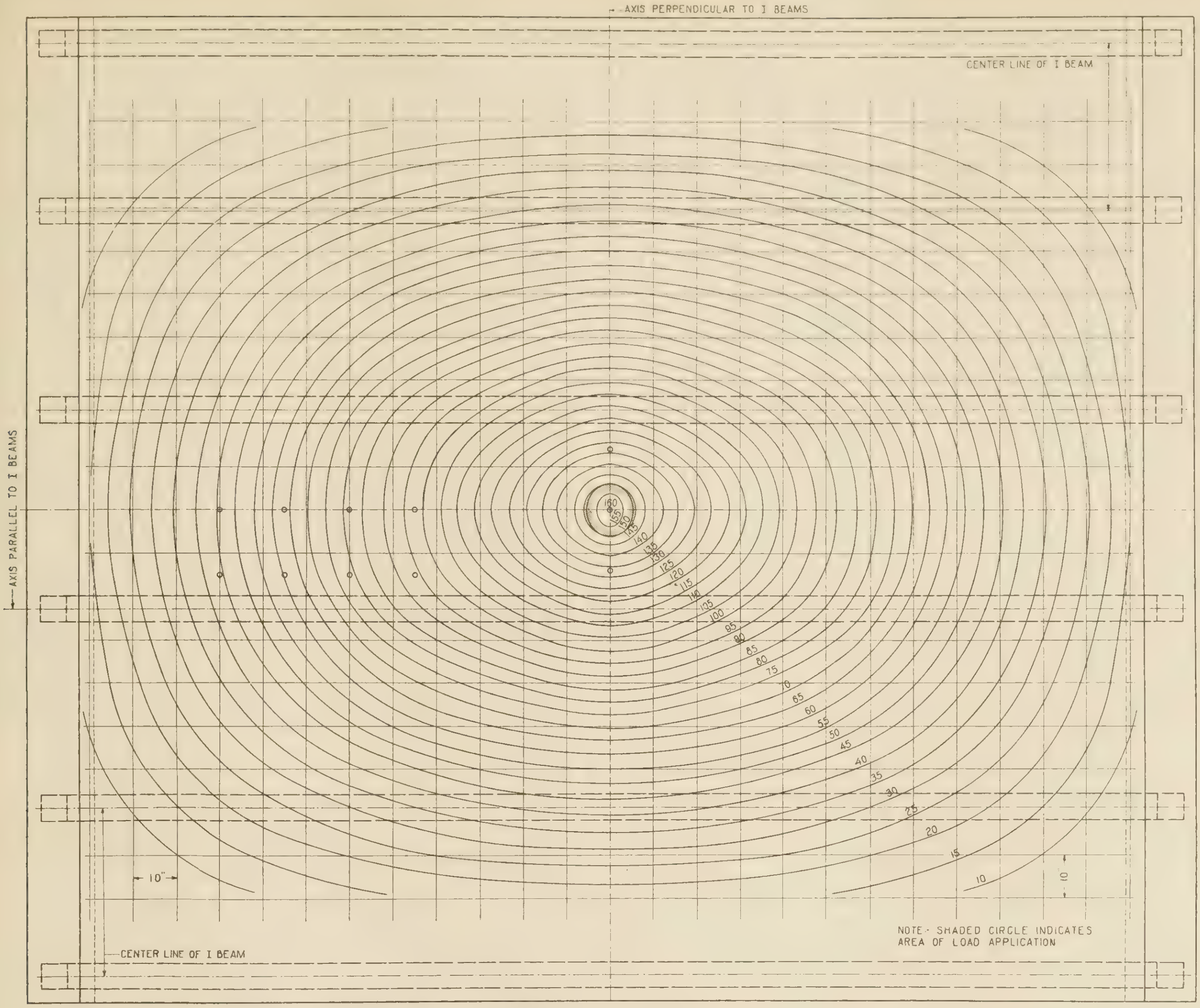


Fig. 43.—Sketch Showing Deflection Contours Produced by a Center Load of 40,000 Pounds, Contour Interval 0.005 Inch

and, as previously remarked, it was necessary to spring them into place and wire them to the forms. On removing the forms there is little doubt that the spring action of these trusses introduced initial stresses in the slab in some cases large enough to partially neutralize or appreciably increase the stresses caused by static loads and impact.

Difficulty was experienced in getting satisfactory readings on all of the large number of gauges used in these tests. It was necessary to allow these gauges to remain in place for comparatively long periods which may have introduced slight errors due to temperature changes or dust on the recording slides.

Tests of the graphic strain gauges, under impact, made at Johns Hopkins University have shown them free from inertia effects and their records to be reliable. Nevertheless, difficulty was experienced in keeping gauge points in the slab from loosening and the glass slides from slipping, especially on the bottom of the slab, when delivering severe impact blows.

The principal value of these measurements is to show the approximate maximum stresses developed under the loads imposed.

Measurements of reactions are shown in Figures 38 and 40. In Figure 39 the variation between the sum of the measured reactions and the load imposed varies from 4 to 28 per cent, the 28 per cent variation being for the low load of 5,000 pounds.

The reactions shown seem to support the contention that there was present a certain arching action in the slab due to the spring action of the steel. It is to be noted that as the load increased the reactions more nearly approach symmetry and the error decreases to a reasonable variation of about 10 per cent. By a comparison of results as shown in Figures 38 and 40 it will be seen that for **I** beams 2, 3, 4 and 5, reactions as measured vary directly as the loads applied within the limits of accuracy of these measurements. The measurements at **I** beams 1 and 6 are so small as to be indeterminate considering the methods used in making the measurements.

A SUGGESTED METHOD OF ANALYSIS

No attempt to develop a possible method of analysis for this type of slab has been made by the Bureau of Public Roads. However, engineers of the Delaware

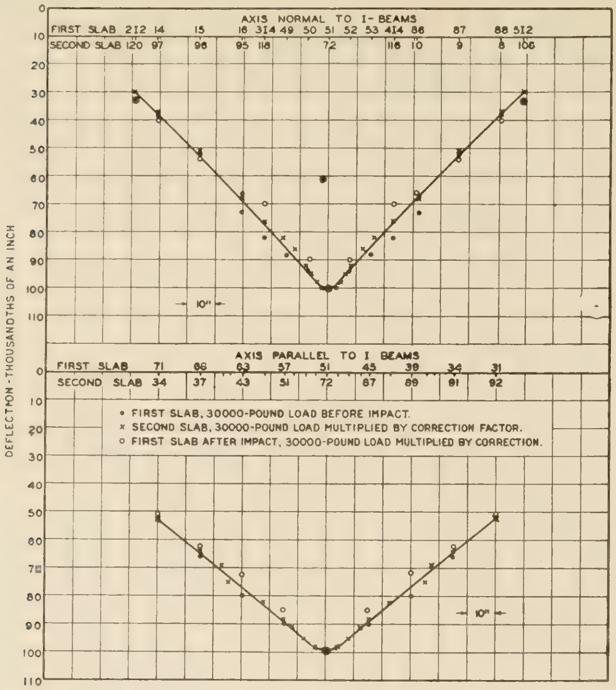


FIG. 44.—COMPARISON OF TYPES OF DEFLECTION CURVES FOR FIRST AND SECOND SLABS. CORRECTION FACTORS FOR DEFLECTION OF FIRST SLAB UNDER STATIC LOADING AFTER IMPACT TESTS AND FOR SECOND SLAB DETERMINED AS EXPLAINED IN TEXT

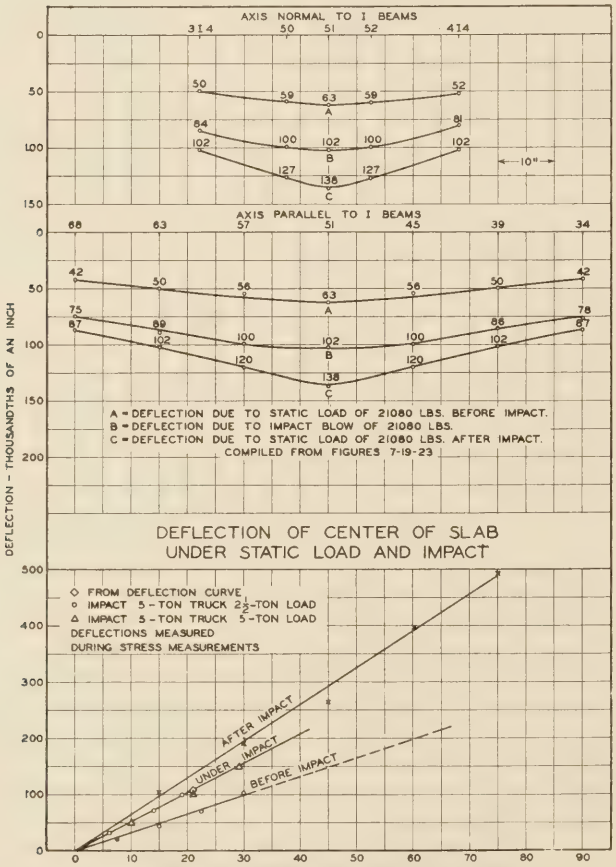


FIG. 45.—COMPARISON OF DEFLECTION OF SLAB UNDER STATIC LOAD AND IMPACT

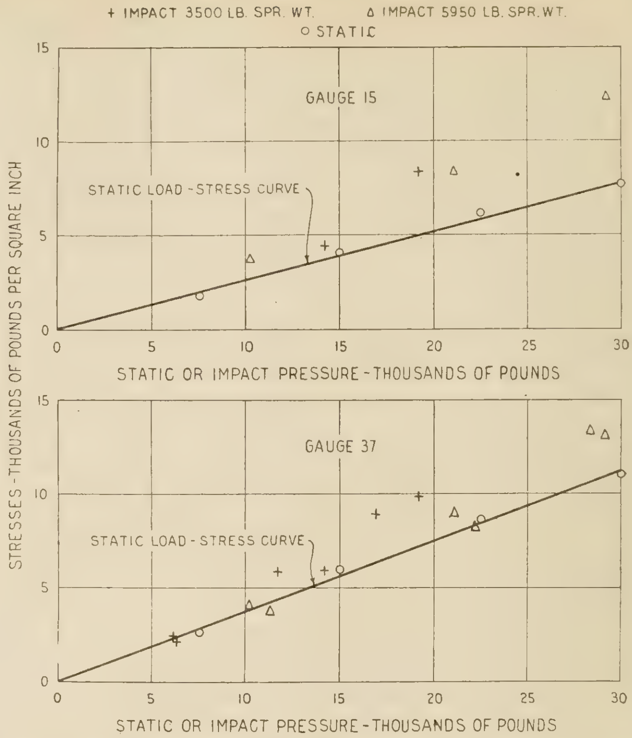


FIG. 46.—COMPARISON OF STRESSES IN REINFORCING TRUSSES OF FIRST SLAB FROM STATIC AMD IMPACT LOADS. (SEE FIGS. 5, 11, AND TABLE 1)

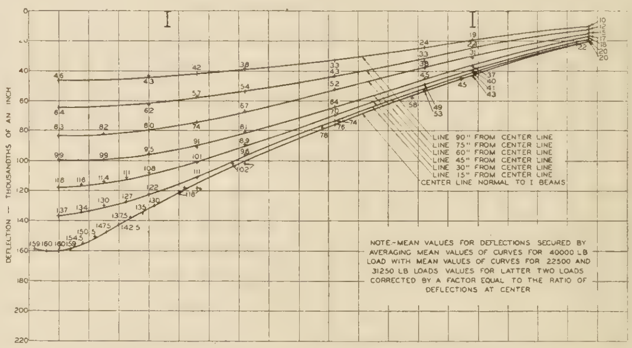


FIG. 47.—COMPOSITE OR TYPE DEFLECTION CURVES ON CENTER LINE NORMAL TO I BEAMS AND AT 15-INCH INTERVALS FOR STATIC LOAD OF 40,000 POUNDS

River Bridge Joint Commission who cooperated in the tests have suggested that analysis be made in the following manner:

By taking the deflections along the axes which are symmetrically located in the slab, composite or type curves may be developed as shown in Figure 47.

By developing graphically successive differentials of the original or type curves, by the tangent or other method, type curves for moment, shear and load distribution may be developed. An example of this for the first slab is shown in Figure 48.

As deflections were measured at shorter intervals along the axes of the second slab and as the deflections were measured by Ames dials fixed in place it is probable that better results would be obtained by using the deflection curves of the second slab. Composite curves for this slab are shown in Figure 47.

(Continued on p. 189)

FURTHER TESTS OF VIBROLITHIC CONCRETE

By the Division of Tests, United States Bureau of Public Roads. Reported by L. W. TELLER, Engineer of Tests, and C. E. PROUDLEY, Assistant Engineer of Tests

COMPARATIVE transverse bending tests on normal and Vibrolithic concrete have recently been completed by the Bureau of Public Roads. The tests were carried on at the Arlington Experimental Station of the bureau and the April, 1926, issue of Public Roads contained a progress report, giving the data obtained as a result of tests of 28-day specimens. Since then the investigation has been concluded with the testing of a duplicate series of one-year specimens. This report is concerned largely with these later tests and includes data obtained in auxiliary investigations which have been made.

Fig. 1.—Slabs and Apparatus at the Beginning of the One-year Tests

EFFORT MADE TO ELIMINATE ALL VARIABLES EXCEPT THOSE BEING STUDIED

The object of the investigation was to obtain data on the resistance to rupture by bending, of specimens of different mixes fabricated by each of the two methods. These tests must not be considered, however, as a complete comparative study of Vibrolithic and normally finished pavements. Only one kind of aggregate was used and there is no evidence as to the effect of other kinds. The normal slabs were finished by hand, and the results obtained apply only to this method of finishing. The effect of machine finishing was not investigated.

In these experiments every effort was made to eliminate all variables except those which were being studied and to have all operations performed under as nearly similar working conditions as possible.

The methods of fabrication and of testing were given in detail in the previous report and will be outlined only briefly here. The test specimens were slabs 36 inches wide, 72 inches long, and 6 inches thick, made of carefully proportioned mixes. The coarse aggregate was limestone from Frederick, Md., graded from 2 inches to ¼ inch. The fine aggregate was clean and fairly coarse Potomac River sand. A carefully blended Portland cement of good quality was used. A correction was made for the bulking of the sand. The consistency for both types of slabs was maintained at about a 2-inch slump, or a flow of 110 to 115 on the 30-inch flow table, using a ⅛-inch drop.

Uniformity of the quality of the cement used throughout the tests was indicated at one year as at 28 days by tests of 1:2 Ottawa sand mortar beams. These mortar tests are shown in Table 3 and Figure 2, and show that the cement was of satisfactory uniformity.

The concrete for all slabs was placed in the forms, tamped and struck off with a straight 2 by 12 inch strike board worked lengthwise over each group of five slabs. The normal finishing consisted of belting lengthwise over the slabs with an 8-inch rubber belt. The Vibrolithic concrete was covered uniformly with the Frederick limestone of 2-inch to 1-inch size at the following rate:

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

The special slatted platforms were placed on this stone and the vibrators were run over them, according to the patented process. The irregular mortar surface left on the removal of the platforms was smoothed down with a long-handled steel float, after which the surface was belted.

CURING OF SLABS AND TRANSVERSE BENDING TESTS

As soon as their hardness would permit, all slabs were covered with wet burlap, kept damp until the following day when they were covered with damp earth. This earth was kept damp for 28 days, after which it was removed and the slabs for testing at 28 days were broken.

The rest of the slabs, i. e., the one-year specimens, were left on the original subgrade during the remainder of the year and were covered with earth and kept wet for the 28 days immediately prior to testing in order that the concrete in all the specimens should be in as nearly a uniform condition of moisture as possible at the time of the test. Figure 1 shows the appearance of the general layout of the test sections at the beginning of the one-year tests.

The test data for the slabs, as given in Tables 1, 2, and 3, were obtained under practically the same conditions as the data which appear in Tables 1, 2, and 3 of the 28-day report.[1] The testing machine provided a span of 60 inches and applied the load at the third points over the entire width of the slab. The load was applied by means of a hydraulic jack and its magnitude was measured by the deflection of a pair of calibrated steel beams. The load causing failure was the sum of three components—the load applied by the jack, the dead load of the knife edges, jack and calibrated beams, concentrated at the points of application, and the uniform distributed load of the slab itself. A refinement in computation was introduced in the one-year tests, however, by using the true weight of each specimen in determining the dead load due to the slab instead of using an average figure for normal concrete and another average for Vibrolithic concrete, as was done previously in the 28-day tests. This refinement resulted in a very small difference in unit stress in most

[1] See Public Roads, vol. 7, No. 2, April, 1926.

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

Mix	Slab No.	Surface in tension	Depth	Section modulus $\frac{I}{c}$	Total load at rupture	Modulus of rupture: Individual slabs	Modulus of rupture: Averages	Strength ratio $\frac{\text{Tension in top}}{\text{Tension in bottom}}$	Variation of individual tests from average of groups	
			Inches	*Inches*³	*Pounds*	*Pounds per sq. in.*	*Pounds per sq. in.*		*Per cent*	
	6	Top	6.28	240.6	16,334	679			7.5	
	9	do	6.09	230.2	16,904	734			0.0	
	17	do	6.24	241.6	19,126	792	734		7.9	3.5
	18	do	6.31	246.5	17,876	725			1.2	
	19	do	6.29	245.0	18,153	741			1.0	
1:1½:3	7	Bottom	6.18	231.1	17,470	756	723	1.029	6.3	
	8	do	6.21	233.7	15,356	657			7.9	
	10	do	6.05	221.1	14,610	661	713		7.3	6.9
	16	do	6.22	253.1	19,448	792			11.1	
	20	do	6.41	247.8	17,320	699			2.0	
	27	Top	6.29	245.0	15,302	625			4.9	
	29	do	6.40	253.7	15,468	610			2.3	
	36	do	6.12	226.4	14,482	640	596		7.4	5.9
	38	do	6.25	242.7	12,672	522			12.4	
	40	do	6.45	253.2	14,723	581			2.5	
1:2:3	26	Bottom	6.32	240.6	18,151	754	613	.945	19.5	
	28	do	6.29	239.6	15,740	657			4.1	
	30	do	6.29	238.7	15,745	660	631		4.6	11.3
	37	do	6.18	231.1	13,010	563			10.8	
	39	do	6.42	249.0	12,988	522			17.3	
	47	Top	5.90	216.3	13,645	599			6.0	
	49	do	6.05	227.3	14,253	627			1.6	
	56	do	6.05	223.3	15,648	701	637		10.0	4.2
	58	do	6.32	247.6	15,862	641			0.6	
	60	do	6.22	236.1	14,624	619			2.8	
1:2:3½	46	Bottom	5.99	216.7	14,318	661	620	1.055	9.4	
	48	do	6.00	218.2	13,089	600			.7	
	50	do	6.20	231.6	13,476	582	604		3.6	5.1
	57	do	6.19	232.2	14,513	625			3.5	
	59	do	6.18	231.1	12,772	553			8.4	
	67	Top	5.99	223.0	13,202	592			1.0	
	69	do	6.04	227.0	13,569	598			0.0	
	76	do	6.01	220.3	14,224	646	598		8.0	3.2
	78	do	5.99	223.0	13,334	598			0.0	
	80	do	6.01	220.3	12,247	556			7.0	
1:2:4	66	Bottom	6.06	221.4	14,518	656	595	1.010	10.8	
	68	do	6.01	219.3	15,605	712			20.3	
	70	do	6.12	225.8	11,855	525	592		11.2	12.4
	77	do	6.00	218.2	12,040	552			6.8	
	79	do	5.98	217.6	11,224	516			12.8	
Average		Top								4.3
		Bottom								8.9

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

Mix	Slab No.	Surface in tension	Depth	Section modulus $\frac{I}{c}$	Total load at rupture	Modulus of rupture: Individual slabs	Modulus of rupture: Average	Strength ratio $\frac{\text{Tension in top}}{\text{Tension in bottom}}$	Variation of individual tests from average of groups	
			Inches	*Inches*³	*Pounds*	*Pounds per sq. in.*	*Pounds per sq. in.*		*Per cent*	
	2	Top	6.18	237.2	20.709	873			0.8	
	4	do	6.28	244.6	19.706	806			8.4	
	11	do	6.02	221.5	21,460	969	880		10.1	4.1
	13	do	6.19	237.6	20,689	871			1.0	
	15	do	6.02	221.5	19.552	883			.3	
1:1½:3½	1	Bottom	6.24	234.6	22,077	941	917	0.922	1.4	
	3	do	6.26	237.0	22,852	964			1.0	
	5	do	6.25	233.4	20,949	897	954		6.0	4.3
	12	do	6.00	218.2	22,800	1,045			9.5	
	14	do	6.12	226.8	20,889	921			3.5	
	22	Top	6.12	232.8	17,335	745			3.6	
	24	do	6.54	264.5	17,987	680			5.4	
	31	do	6.14	230.2	15,914	691	719		3.9	6.6
	33	do	6.18	237.2	19,241	811			12.8	
	35	do	6.20	234.7	15,628	666			7.4	
1:2:3½	21	Bottom	5.92	212.6	18,758	882	782	0.850	4.3	
	23	do	6.22	234.0	21,369	913			7.9	
	25	do	6.28	237.6	20,505	863	846		2.0	5.7
	32	do	6.20	232.6	17,297	744			12.1	
	34	do	6.09	225.0	18,582	826			2.4	
	42	Top	5.98	222.0	15,970	719			1.8	
	44	do	5.86	217.0	15,225	713			2.6	
	51	do	6.28	240.6	18,115	753	732		2.9	2.3
	53	do	6.12	232.8	17,486	751			2.6	
	55	do	6.12	228.7	16,504	722			1.4	
1:2:4	41	Bottom	6.18	230.2	17,101	743	748	0.957	2.9	
	43	do	6.02	220.4	17,551	796			4.1	
	45	do	5.91	211.1	15,986	757	765		1.0	3.2
	52	do	6.14	228.2	18,146	795			3.9	
	54	do	6.22	234.0	17,176	734			4.1	
	62	Top	6.11	231.7	13,927	601			2.1	
	64	do	6.39	252.6	14,277	565			8.0	
	71	do	6.12	228.7	13,532	592	614		3.6	5.5
	73	do	6.11	231.7	15,256	658			7.2	
	75	do	6.12	228.7	14,944	653			6.4	
1:2:4½	61	Bottom	6.20	231.6	18,783	811	673	0.840	10.9	
	63	do	6.21	233.7	15,915	681			6.8	
	65	do	6.29	230.9	16,932	730	731		0.1	5.0
	72	do	6.01	219.3	16,293	743			1.6	
	74	do	6.11	226.4	15,658	691			5.5	
Average		Top								4.6
		Bottom								4.5

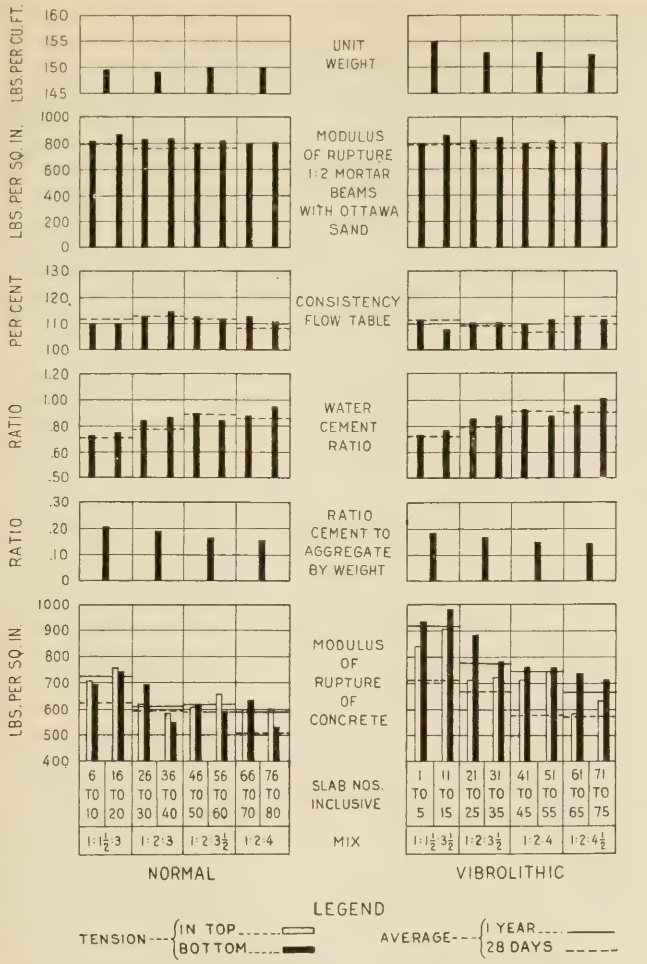


FIG 2.—RESULTS OF TESTS OF NORMAL AND VIBROLITHIC CONCRETE AND MORTAR BRIQUETTES AND BEAMS

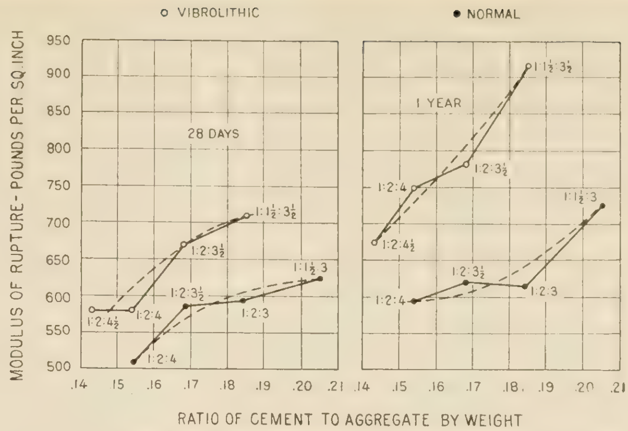


FIG. 3.—AVERAGE MODULUS OF RUPTURE COMPARED WITH CEMENT-AGGREGATE RATIO

TABLE 3.—*Summary of all test data by groups on normal and Vibrolithic concrete*

Finish	Normal concrete							
Mix	1:1½:3		1:2:3		1:2:3½		1:2:4	
Slab Nos., inclusive	6 to 10	16 to 20	26 to 30	36 to 40	46 to 50	56 to 60	66 to 70	76 to 80
Tension in top at rupture, pounds per square inch	707	753	618	581	613	654	595	600
Tension in bottom at rupture, pounds per square inch	691	746	690	543	614	589	631	534
Ratio of cement to aggregate by weight	0.205		0.184		0.168		0.154	
Water-cement ratio	0.73	0.75	0.85	0.87	0.90	0.85	0.88	0.95
Consistency, flow table	110	110	113	115	113	112	113	111
Modulus of rupture, pounds per square inch, 1:2 mortar using Ottawa sand	809	864	829	842	802	818	793	810
Weight per cubic foot	149.3		148.6		150.1		149.9	
Finish	**Vibrolithic concrete**							
Mix	1:1½:3½		1:2:3½		1:2:4		1:2:4½	
Slab Nos., inclusive	1 to 5	11 to 15	21 to 25	31 to 35	41 to 45	51 to 55	61 to 65	71 to 75
Tension in top at rupture, pounds per square inch	840	908	713	723	716	742	583	635
Tension in bottom at rupture, pounds per square inch	934	983	886	785	765	765	741	717
Ratio of cement to aggregate by weight	0.185		0.168		0.154		0.143	
Water-cement ratio	0.74	0.77	0.86	0.88	0.92	0.88	0.96	1.01
Consistency, flow table	112	108	111	111	110	112	113	112
Modulus of rupture, pounds per square inch 1:2 mortar using Ottawa sand	809	864	829	842	802	818	793	810
Weight per cubic foot	155.2		152.9		153.0		152.6	

cases. Figure 2 gives essentially the same data as shown in Table 3 and in addition, however, the averages of the 28-day test data are indicated by horizontal dotted lines.

MODULUS OF ELASTICITY DETERMINED FROM DATA SECURED IN BENDING TEST

At the same time that the one-year tests were made certain slabs were selected for determination of the maximum unit fiber deformation and modulus of elasticity. This was done by means of graphic strain gauges[2] set 0.25 inch from the surface of the slab between suitable gauge points. The gauges were placed at the top and bottom at each edge of the slab at the center of the span. Readings of these four gauges for progressive increments of load were the basis of computations of unit fiber deformations which were plotted with the computed unit fiber stresses. The initial modulus of elasticity was determined from these graphs. Figure 4 shows a typical graph of these data. Two slabs of each mix for both methods of finishing were tested in this manner, one being tested with the finished side in tension and the other with the subgrade side in tension. The values thus obtained are given in Table 4.

COMPRESSION TESTS MADE ON FROZEN AND UNFROZEN CORES FROM SLABS

Cores were drilled from the fragments of 28-day specimens at about six months for compression tests and for alternate freezing and thawing. Results of these tests warranted the drilling of more cores from the one-year slabs and a repetition of the comparative compression and freezing tests. The cores were taken with a Calyx core drill and were 6 inches in diameter and approximately 6 inches high.

Shortly after the 28-day slab cores were drilled, an absorption test was made on each one. They were dried in an oven at about 80° C., cooled, weighed, immersed in water for 48 hours at 40° C. and again weighed. The results of these determinations are given in Table 5.

[2] GOLDBECK, A. T., POCKET STRAIN GAUGE GIVES STRESSES IN CONCRETE ROADS Engineering News-Record, Mar. 29, 1923

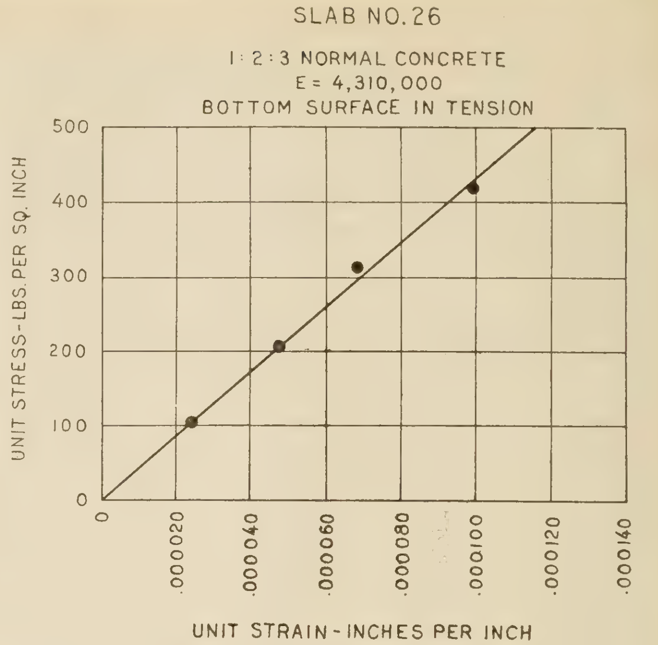
Table 4.—*Modulus of elasticity determined by the transverse bending of slabs and by the compression of drilled cores* [1]

Normal					Vibrolithic				
Mix	Slabs		Cores		Mix	Slabs		Cores	
	No.	Modulus	No.	Modulus		No.	Modulus	No.	Modulus
		Pounds per sq. in.		*Pounds per sq. in.*			*Pounds per sq. in.*		*Pounds per sq. in.*
	6	3, 550, 000	6	3, 200, 000		5	5, 040, 000	----	5, 980, 000
1:1½:3	16	3, 850, 000	10	5, 020, 000	1:1½:3½	15	5, 250, 000	2	5, 260, 000
	---	-------	18	4, 500, 000		---	-------	11	6, 944, 000
Average	----	3, 700, 000	----	4, 240, 000	Average	----	5, 140, 000	----	6, 061, 000
	26	4, 310, 000	28	4, 640, 000		25	3, 830, 000	21	2, 960, 000
1:2:3	36	3, 320, 000	29	4, 720, 000	1:2:3½	35	4, 340, 000	31	4, 500, 000
	---	-------	39	3, 860, 000		---	-------	32	3, 470, 000
Average	----	3, 810, 000	----	4, 410, 000	Average	----	4, 080, 000	----	3, 985, 000
	46	3, 100, 000	50	5, 280, 000		45	4. 420, 000	42	4, 400, 000
1:2:3½	56	4, 150, 000	59	3, 080, 000	1:2:4	55	2, 920, 000	43	5, 930, 000
	---	-------	60	3, 380, 000		---	-------	55	4, 100, 000
Average	----	3, 620, 000	----	3, 913. 000	Average	----	3, 670, 000	----	4, 810, 000
	66	4, 680, 000	69	4, 720, 000		65	4, 700, 000	61	5, 600, 000
1:2:4	76	2, 970, 000	70	6, 000, 000	1:2:4½	75	3, 630, 000	62	4, 300, 000
	---	-------	80	5, 620, 000		---	-------	71	4, 410, 000
Average	----	3, 820, 000	----	5, 447, 000	Average	----	4, 160. 000	----	4, 770, 000

[1] Of the normal group slabs 16, 26, 46, and 66 were tested with the bottom surface in tension and slabs 6, 36, 56, and 76 were tested with the top surface in tension. Of the Vibrolithic group slabs 5, 25, 45, and 65 were tested with the bottom surface in tension and slabs 15, 35, 55, and 75 were tested with the top surface in tension.

Table 5.—*Absorption tests on normal and Vibrolithic concrete a age of 7 months*

Normal				Vibrolithic			
Mix	Slab No.	Absorption	Average absorption	Mix	Slab No.	Absorption	Average absorption
		Per cent	*Per cent*			*Per cent*	*Per cent*
1:1½:3	88	3. 3	4. 1	1:1½:3½	83	3. 1	2. 5
		3. 5				3. 6	
	89	2. 8			84	1. 8	
		3. 2				1. 1	
	99	4. 4			91	2. 4	
		4. 8				2. 4	
	100	4. 4			93	4. 1	
		6. 1				1. 3	
1:2:3	118	2. 0	1. 7	1:2:3½	111	2. 6	2. 3
		1. 3				2. 7	
	119	1. 7			112	3. 2	
		. 9				1. 0	
	167	1. 1			163	3. 3	
		4. 0				3. 9	
	169	1. 0			164	. 9	
		1. 8				1. 0	
1:2:3½	128	3. 3	2. 2	1:2:4	122	4. 3	3. 1
		1. 8				4. 2	
	129	1. 6			123	2. 4	
		1. 6				1. 4	
	138	. 9			131	4. 0	
		3. 9				1. 0	
	140	. 8			133	4. 6	
		3. 9				3. 0	
1:2:4	149	3. 7	2. 0	1:2:4½	141	1. 0	1. 6
		. 9				4. 4	
	150	1. 4			142	. 8	
		1. 2				1. 1	
	158	2. 5			151	. 9	
		1. 1				3. 1	
	160	2. 4			153	. 7	
		2. 4				1. 1	
Average	------	-------	2. 5	Average	------	-------	2. 4



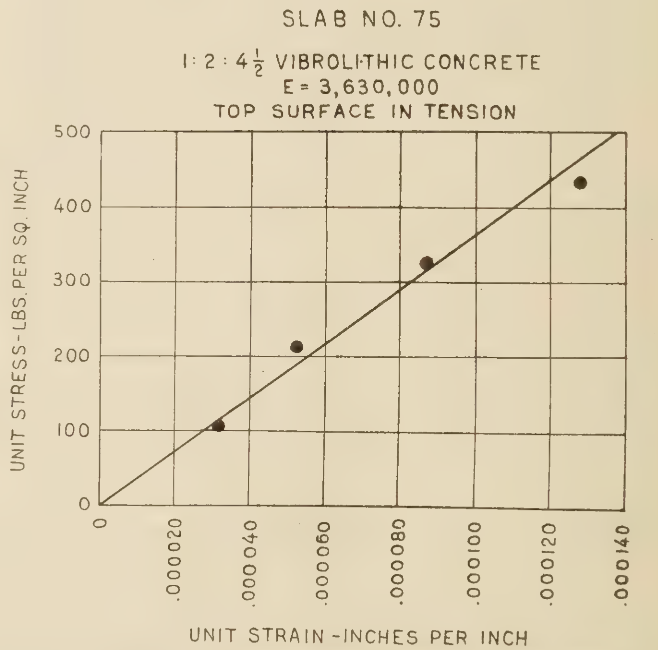


Fig. 4.—Typical Stress-strain Curves from the Transverse Bending Tests

In the first series of freezing tests, three cores of each mix of both normal and Vibrolithic concrete were immersed in water in containers and surrounded by brine at approximately −12° C. When frozen solid the containers were removed from the brine and permitted to thaw. After 10 alternations of such freezing and thawing, all of the specimens showed disintegration so they were thawed out, capped and tested in compression.

In the second series of freezing tests the specimens were kept saturated but not immersed during freezing. This change was made as an experiment, as it was thought that the rapid disintegration in the first test might be partially due to the expansion of the surrounding ice during freezing.

Unfortunately, due to leakage that occurred in some of the containers during the course of these later tests, brine entered some of the containers and came in contact with the lower end of some of the cores. It was impossible to determine with certainty which of the specimens had been exposed to the brine so that the effect of such exposure is not known. In view of this, none of the data obtained is above suspicion and is not included. However, even with this uncertainty admitted it can be said that it took approximately twice as long to produce disintegration by this method of freezing as by the first method.

The cores which were not frozen, of which there were five in the first series for each mix and kind of concrete, were left in the field near the slabs from which they were drilled until they were about 12½ months old, at which time they were prepared for testing by immersing until a uniform condition of moisture was obtained. From the one-year slabs, three cores of each mix and kind were drilled, capped, and saturated with water and tested at the age of 17 months. Tables 6 and 7 give the test results on the unfrozen and frozen specimens, respectively, while in Table 8 are shown the data from the compression tests at 17 months.

TABLE 6.—*Compression tests on 6 by 6 inch cores drilled from normal and Vibrolithic concrete at age of 13 months*

Normal concrete cores				Vibrolithic concrete cores			
Mix	Slab No.	Load at failure	Average load at falure	Mix	Slab No.	Load at failure	Average load at failure
		Pounds per sq. in.	*Pounds per sq. in.*			*Pounds per sq. in.*	*Pounds per sq. in.*
	88	5, 527			83	4, 800	
	89	5, 340			84	5, 580	
1:1½:3	99	4, 985	4, 869	1:1½:3½	84	4, 882	5, 047
	99	3, 707			91	4, 580	
	100	4, 788			93	5, 391	
	118	4, 377			111	5, 142	
	118	5, 382	5, 185		112	5, 673	
1:2:3	119	5, 264		1:2:3½	112	4, 856	5, 322
	167	[1] 5, 620			163	5, 370	
	169	5, 280			164	[1] 5, 570	
	128	4, 360			122	3, 768	
	129	4, 378			123	5, 381	
1:2:3½	138	5, 710	4, 770	1:2:4	123	4, 640	4, 468
	140	3, 823			131	4, 561	
	140	5, 578			133	3, 988	
	149	5, 477			141	4, 400	
	150	4, 478			142	5, 259	
1:2:4	158	4, 711	5, 223	1:2:4½	151	4, 600	4, 900
	160	[1] 5, 780			153	5, 225	
	160	5, 670			153	5, 014	
Average			5, 012	Average			4, 934

[1] Total load over 200,000 pounds.

TABLE 7.—*Compressive strength of 6 by 6 inch drilled cores after 10 alternations of freezing and thawing at age of 8 months*

Normal concrete cores				Vibrolithic concrete cores			
Mix	Slab No.	Load at failure	Average load at failure	Mix	Slab No.	Load at failure	Average load at failure
		Pounds per sq. in.	*Pounds per sq. in.*			*Pounds per sq in.*	*Pounds per sq. in.*
	88	2, 730			83	2, 025	
1:1½:3	89	3, 010	2, 653	1:1½:3½	91	2, 735	2, 047
	100	2, 220			93	1, 380	
	167	2, 850			111	4, 380	
1:2:3	169	4, 840	3, 427	1:2:3½	163	2, 842	4, 211
	119	2, 590			164	5, 410	
	128	4, 020			122	2, 475	
1:2:3½	129	3, 680	3, 562	1:2:4	131	4, 275	2, 715
	138	2, 985			133	1, 396	
	149	2, 190			141	2, 770	
1:2:4	150	4, 470	3, 645	1:2:4½	142	3, 970	4, 027
	158	4, 275			151	5, 340	
Average			3, 322	Average			3, 250

TABLE 8.—*Compression tests on 6 by 6 inch drilled cores from normal and Vibrolithic concrete at age of 17 months*

Normal concrete cores				Vibrolithic concrete cores			
Mix	Slab No.	Load at failure	Average load at failure	Mix	Slab No.	Load at failure	Average load at failure
		Pounds per sq. in.	*Pounds per sq. in.*			*Pounds per sq. in.*	*Pounds per sq. in.*
	9	5, 593			1	5, 465	
1:1½:3	10	4, 993	5, 459	1:1½:3½	2	5, 496	5, 988
	18	5, 790			11	7, 004	
	28	5, 849			21	5, 728	
1:2:3	29	5, 770	5, 562	1:2:3½	31	4, 809	5, 286
	39	5, 067			32	5, 320	
	50	5, 059			42	5, 105	
1:2:3½	59	4, 492	5, 111	1:2:4	43	4, 573	4, 995
	60	5, 783			55	5, 306	
	69	4, 536			61	5, 490	
1:2:4	70	3, 262	4, 311	1:2:4½	62	4, 378	4, 773
	80	5, 136			71	4, 451	
Average			5, 111	Average			5, 261

As a check on the modulus of elasticity of the concrete in the slabs as determined by the strain gauge measurements, tests to determine the modulus of elasticity in compression were made on each of the unfrozen cores from the one-year slabs prior to compression test. This was done by means of a mirror extensometer [3] and the results are given in Table 4 where a comparison may be readily made.

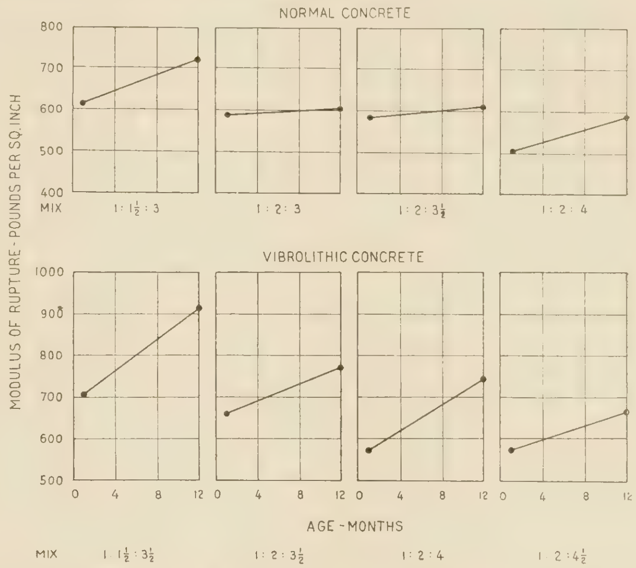


FIG. 5.—DIAGRAM SHOWING INCREASE IN MODULUS OF RUPTURE FROM 28 DAYS TO ONE YEAR FOR VARIOUS MIXES OF NORMAL AND VIBROLITHIC CONCRETE

It was thought that if the true section modulus of the slabs could be determined for those slabs which were honeycombed (for a discussion of the causes of honeycombing see the report on the 28-day tests) the computed strength would be more nearly the true average than when determined by the section modulus computed for a solid area. To study this effect half-size drawings of the broken section of the slabs in question were made by two observers working independently, and the areas of doubtful density carefully measured and located on the sketch. These are shown in Figure 6, the cross sections being shown in the position in which they were tested.

The area of these spaces and their distance from the neutral axis, the location of which was determined by a mechanical integrator for this part of the study, furnished data for a correction of the moment of inertia of the slab. Thirty-three of the most noticeably honeycombed specimens of both types of concrete were selected for investigation, and the results of the stress computations, although not published with this report, are discussed later.

TEST DATA DISCUSSION

A study of Figure 2 shows some interesting relationships between slab strength, water-cement ratio, consistency, and unit weight. As the water-cement ratio increases the slab strength decreases and the corresponding changes are in accord with the generally accepted theory in regard to this ratio. In the normal slabs, the richest mix with relatively high strength has

[3] JOHNSON, A. N., DIRECT MEASUREMENT OF POISSON'S RATIO FOR CONCRETE. Proceedings A. S. T. M., vol. 24, Part II, p. 1024. 1924.

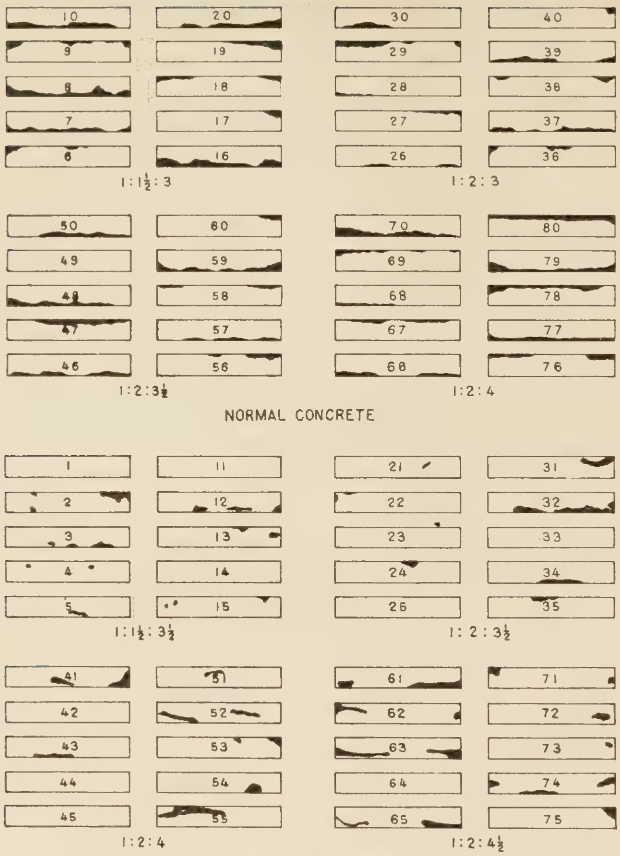


Fig. 6.—Areas of Questionable Density in Test Slabs, as Averaged from the Sketches of Two Independent Observers. The Slabs Are Shown in the Position in Which They Were Tested

a water-cement ratio of about 0.74. The remaining three mixes are comparatively close on strength and the water-cement ratios are 0.86, 0.87 and 0.91 which, from the standpoint of practical application are not greatly different. In the Vibrolithic specimens, the range in strength is greater as is also the range in water-cement ratio.

In comparing the consistencies it should be repeated that the concrete was mixed with the intention of having a flow of 110 to 115. This average was maintained with the exception of the 1 : 1½ : 3½ concrete used for slabs 11 to 15, inclusive. Here, in spite of a slightly higher water-cement ratio the slab strength is greater than the corresponding slabs 1 to 5. A similar condition is evident between the 1 : 2 : 3 and 1 : 2 : 3½ mixes of normal concrete. It may be that the measured consistency is more nearly an index of the actual water-cement ratio in the mixed concrete than can be obtained from a computation using the amount of water, moisture in the sand and volume of cement introduced into the mixer drum. High temperatures existing at the time these slabs were laid probably caused an unusually high rate of evaporation with consequent lower water-cement ratio and dryer consistency.

The unit weight determinations (Table 3) show that the addition of top stone in the Vibrolithic slabs resulted in a concrete of higher weight per cubic foot.

The quality of the cement as indicated by tests of the 1 : 2 mortar beams at one year was quite uniform, the maximum variation from average strength being 5.2 per cent and the average was 2.2 per cent. The test beams were 2 inches wide, 3 inches deep and 18 inches long, and were tested on a 15-inch span with center loading. Five specimens constituted a set.

STRENGTH OF VIBROLITHIC AND NORMAL CONCRETE SLABS COMPARED

The above discussion refers merely to the concordance of the slab strengths with other data relating to the mixtures. A comparison of the normal concrete with the Vibrolithic concrete may be made from Figure 2 or Table 3, but is more readily seen by referring to the dotted average curves in Figure 3. Considering first the strengths obtained for equal quantities of cement per unit volume of aggregate, the ordinates corresponding to the abscissa 0.154 representing a 1 : 2 : 4 normal concrete shows Vibrolithic concrete to be approximately 123 per cent of the normal concrete strength and in the richer mix at 0.184 the strength ratio is about 143 per cent. The corresponding ratios on the 28 day curves of Figure 3 are approximately 120 per cent and 117 per cent, respectively.

A more practical comparison is that showing the amount of cement required to give equal strength. Taking a modulus of rupture of 700 pounds per square inch for the comparison, the cement-aggregate ratio for Vibrolithic concrete is about 0.148 and for normal concrete is 0.201, an increase of about 36 per cent in cement requirement for the normal concrete. Or, expressed as a percentage of the normal requirement for concrete, the Vibrolithic concrete is equal in strength when 74 per cent of the cement required for normal concrete is used. For a modulus of rupture of 600 pounds per square inch at 28 days, the corresponding figures were 19 per cent and 84 per cent.

With respect to the relation between the tensile strength of the concrete in the top and in the bottom surfaces of the slabs of the two types, it will be noted that a change occurred between the 28-day and the 1-year tests. The earlier tests showed that in normal concrete the tensile strength in the bottom of the slab was only 87.7 per cent of that in the top and in Vibrolithic concrete it was 98 per cent. At one year, however, the resistance to tension in the bottom of the normal concrete specimens was 99 per cent of that in the top, whereas in the Vibrolithic concrete the resistance to tension in the bottom is 112 per cent of that in the top. In other words, although the resistance to tension in the bottom of the normal concrete slabs was considerably less than for the top of the slabs at 28 days, the results at one year show practically equal strength in top and bottom. The Vibrolithic concrete which showed practically equal resistance to tension in top and bottom at 28 days was considerably stronger in tension in the bottom fibers than in the top. A possible explanation of the more rapid growth in strength of the bottom of the slabs than occurred in the top for both normal and Vibrolithic is that the bottom being next to the subgrade had more water readily available for continuous hydration of the cement than the top which was exposed to the drying effect of the atmosphere for a period of 10 months.

EFFECT OF HONEYCOMBING DISCUSSED

The above comparisons are based on the averages of all specimens tested even though some varied considerably from the average. The majority of the specimens showing a considerable variation were above the average strength and, as will be brought out in later discussion, it is probable that the strengths obtained are generally as accurate an indication of the strength of these slabs as those of apparently more consistent strength. These values, therefore, have been included in the average just as was done in the report of 28-day tests. Except for three or four of the normal concrete slabs which were tested with tension in the bottom, the percentage variation in both normal and Vibrolithic specimens averages about the same, indicating fairly satisfactory agreement in the slab tests as a whole. It also indicates that the product of either the Vibrolithic process or hand finishing is reasonably uniform in this investigation except for the few instances noted.

In the report of the earlier tests it was pointed out that those slabs whose cross sections were reduced by void spaces, or honeycombing in general, showed the lower modulus of rupture values. This same condition exists in the one-year specimens, as may be seen in the outline drawings of the sections in Figure 6. These outlines are the average result of independent sketches by two observers, and are not claimed to be exact, the solid portions merely representing areas of questionable density. They serve, however, to assist in explaining some of the apparently erratic slab strengths seen in Tables 1 and 2.

For example, slabs Nos. 8 and 10 exhibit relatively greater honeycombing in the bottom than do Nos. 7 or 20, and the strengths of Nos. 8 and 10 are lower than in the other two. Slab No. 16 showed the highest strength in this group tested with bottom in tension, and in fact, is equal to No. 17 which is apparently quite dense, yet inspection reveals the highest percentage of voids in any of the 1 : 1½ : 3 normal concrete slabs. This inconsistency might be considered sufficient reason to exclude it from the average.

In the 1 : 2 : 3 concrete slabs No. 39 is the only specimen which seems to be seriously honeycombed and it is seen has the lowest strength of this group. Slab No. 37 is sufficiently porous to be mentioned and is the next lowest in modulus of rupture. An inconsistency exists in this group also, slab No. 38 showing as low a strength as any but being apparently quite dense.

The next group, 1 : 2 : 3½ normal concrete, has slabs Nos. 50, 48, 47, 46, and 59 noticeably honeycombed. No. 46, which shows the highest strength of those tests with the bottom in tension, exhibits a fair amount of void space at the bottom and might be considered somewhat inconsistent. The other specimens are fairly well in accord with respect to strength and apparent density.

The leanest of the normal concrete, which might be expected to be less dense than the rest, is considerably honeycombed in slabs Nos. 70, 77, 78, 79, and 80. Of the slabs mentioned, No. 79 is the lowest in strength, No. 70 is nearly as low, and No. 78 has a strength equal to the average of the group. Slabs Nos. 80 and 77 are below the average of the group. No. 76 which was fairly dense shows quite high strength, as does also No. 68 which has only a thin line of questionable area on its lower face.

In the Vibrolithic concrete specimens very little difference in density is noticeable until slab No. 32 of the 1 : 2 : 3½ mix is reached. This slab is the lowest in strength of those tested with tension in the bottom. With the exception of this slab, it will be noticed in the 1 : 1½ : 3½ and 1 : 2 : 3½ mixes that the highest percentage variations from the average are due to strengths greater than the average.

The 1 : 2 : 4 mix is not so uniform in density, especially Nos. 41 and 55. The effect, however, of this condition is not great and several inconsistencies could be pointed out but the maximum percentage variation in this group being only 4.1 per cent, the slabs are probably typical and the results reasonably correct. It is most readily apparent in this group that the Vibrolithic process tends to move the void spaces upward, which removes these less dense areas from the region of the maximum stress. This characteristic of Vibrolithic concrete was pointed out in the report of tests on 28-day specimens.

The next group of slabs is of 1 : 2 : 4½ mix and is quite inconsistent with regard to the density-strength comparison. More specifically, No. 64 exhibits practically no voids but has low strength, and No. 61, which seems to be badly honeycombed, had the highest strength of the group. Nos. 63 and 74 have comparatively large areas of poor density and are also comparatively low in strength. Further comparisons of a less striking nature may be made which would be essentially the same as those given above.

MODULUS OF RUPTURE AT 28 DAYS AND ONE YEAR

The change in the modulus of rupture of the various groups of slabs from 28 days to one year, as shown in Figure 5, is also of interest. As an indication of the growth in strength of the concrete, the average of all one-year strength tests of normal concrete specimens was compared with the average of all of the 28-day normal specimens and found to be about 10 per cent greater. Similar comparison of the Vibrolithic concrete shows the one-year specimens to average about 23 per cent greater than the 28-day specimens. The increase in strength of the various mixes is also shown in Figure 3.

The water-cement ratio was slightly higher for the specimens for the 1-year tests than it was for those for the 28-day tests. The average difference was an increase of $0.85-0.81=0.04$ for the normal concrete and an increase of $0.88-0.83=0.05$ for the Vibrolithic concrete. This would tend to make the average increase from 28 days to 1 year slightly less for both normal and Vibrolithic concrete than would be expected had the same water-cement ratios obtained for both test groups.

Although the data on the determination of the value of the modulus of elasticty are somewhat meager, yet there are some interesting indications in them. In general, the values range from 3,000,000 to 6,000,000 pounds per square inch for each type of concrete and seem to bear no relation to either the compressive strength or the modulus of rupture. The average values for E, determined from the strain gauge readings on the flexed beams, are slightly lower than the average values determined by compression on drilled cores. This is true for both types of concrete. Also the average value of E for the normal concrete is somewhat lower than the average value for the Vibrolithic, whether the determination is made by bending or by compression tests.

RESULTS FROM ABSORPTION, FREEZING, AND COMPRESSION OF CORES DISCUSSED

The absorption of the concrete as shown in Table 5 furnishes but little material for definite comment when considered alone. Without going deeply into a discussion of effect of voids on absorption it would seem that other things being equal the richer concrete would have the lower percentage of voids. Also, it is reasonable to suppose that for concrete of identical materials, quantities, and mixing, that which is rodded or vibrated the most will probably give the lower absorption. These two effects have probably offset each other and made the average absorption for the normal concrete and the Vibrolithic concrete practically the same.

In the first series of freezing tests, disintegration was so rapid that no difference in the behavior of the two types of concrete could be noticed. As previously remarked, it was thought that the high compressive stress set up in the specimen by the expansion of the surrounding block of ice caused sufficient differential in deformation of the mortar and stone, each having a different modulus of elasticity, to destroy the bond. The appearance of the frozen specimens indicated such action. Table 7 shows the strengths obtained when these frozen cores were tested in compression. The values are so erratic as to mean practically nothing. When these cores were frozen disintegration began at the corners and this left rounded irregular ends which, even when carefully capped, made the results of compression tests of doubtful value. It is thought that the effect of freezing on the strength of concrete will be indicated far more precisely by specimens which are suitable for transverse bending tests.

As already stated, the second series of freezing tests was unsuccessful so that the data which it was planned to obtain on the relative behavior of the two types of concrete under conditions of alternate freezing and thawing did not materialize.

Cores drilled from the 28-day test slabs were tested in compression at the age of 13 months, and these data are shown in Table 6. Considerable variation is to be noted in the individual test results; the different mixes do not show the strength variations which would be expected, and the difference in strength between the normal and Vibrolithic specimens so consistently indicated in the bending tests does not appear at all in these compression tests. The inference is that compression tests on these drilled cores are not a good measure of the strength of the concrete and indicate little except that, in general, the concrete is of good quality. This is also true of the cores tested at 17 months, the data for which appear in Table 8, except that the average strength of the different mixes appears to be somewhat more in accord with what might be expected.

CONCLUSIONS

When drawing conclusions from the data obtained in this investigation the following fact should be borne in mind. The normal slabs were too dry for hand finishing and the high percentage of honeycombing found in them is the result of this condition. Had the mix of these slabs been wetter a higher degree of compaction would probably have been obtained, but it is probable that the gain in strength which might have been expected as the result of higher density would have been offset by the reduction in strength due to the increase in water content. On the other hand, the dry mix was well designed for machine finishing, and if machine methods had been employed the normal slabs would probably have been more uniform and dense, and their strength correspondingly higher. The difference between the two types of concrete would then have been less. In spite of this, however, an examination of the strength values for the normal concrete for any given age and mix will show that this concrete was of at least average strength.

In summarizing the results of this investigation, it may be said that:

1. For a given cement content the slabs constructed by the Vibrolithic method showed greater strength than those of normal concrete, when measured by the transverse bending test.
2. The Vibrolithic concrete slabs were somewhat more uniform in strength than those of normal concrete in both the 28-day and 1-year tests, both types showing greater uniformity at 1-year than at 28 days.
3. The specimens of Vibrolithic concrete showed a greater percentage of increase in the modulus of rupture from 28 days to 1 year than did those of normal concrete.
4. Absorption tests on 6 by 6 inch drilled cores indicate that there is practically no difference in the absorptive properties of the two types of concrete.
5. Compression tests on 6 by 6 inch drilled cores tested at the ages of 13 and 17 months do not show the difference in strength between normal and Vibrolithic concrete that is indicated by the transverse bending tests.
6. The value of the modulus of elasticity ranges from 3,000,000 to 6,000,000 pounds per square inch for both types of concrete. The average value of E is somewhat higher for the Vibrolithic concrete than for the normal concrete by both the bending and compression test determinations.

APPLICATION OF THE RESULTS OF THE INVESTIGATION IN THE DESIGN OF PAVEMENTS

It has been shown in this report that there was a difference in the modulus of rupture or resistance to bending of concrete placed by the two methods. The magnitude of this difference for the various mixes may be ascertained by comparison of Tables 1 and 2, and the graphical presentation of the same data in Figure 3. Corresponding data will be found in the 28-day report.[4]

For the two comparable mixes the difference may be expressed as a ratio with the following result:

Mix	Bending strength ratio = $\frac{\text{Average modulus of rupture (Vibrolithic)}}{\text{Average modulus of rupture (normal)}}$	
	28-day tests	1-year tests
1 : 2 : 3½	$\frac{669}{586}=1.14$	$\frac{782}{620}=1.26$
1 : 2 : 4	$\frac{579}{507}=1.14$	$\frac{748}{595}=1.26$

It can not be said that the difference found in these tests will hold for concrete constructed of other aggregates. But for the moment, let it be assumed that we expect to use such aggregates as were used in the Arlington tests and that differences in strength such as were found will be obtained with the two methods of placing. How then, can the data developed by this investigation be used in the design of pavements? From the data at hand we may design pavements of equivalent bending strength, by either of the two methods, if we use—

(*a*) the same mix but change the cross-section, or
(*b*) the same cross-section but change the mix.

[4] See Public Roads, vol. 7, No. 2, April, 1926, pp. 38 and 41.

Employing the first method, the knowledge of the modulus of rupture of concrete of a given mix finished by the two methods supplied by this investigation may be used to determine the required thickness of each type by means of the so-called corner formula.

The use of this simple formula for the design of road slabs is based upon the assumption that the most severe condition of loading likely to occur is that of a load concentrated at the extreme corner of a rectangular panel of the slab when, for one reason or another, the subgrade offers practically no support to the corner.

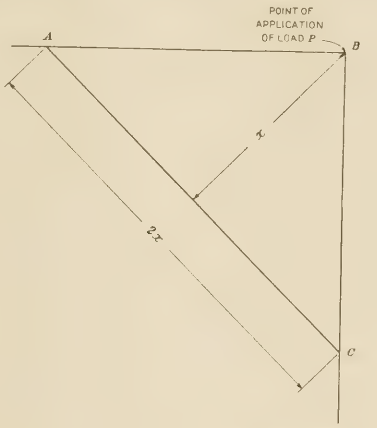


FIG. 7.—DIAGRAM FOR USE WITH DEMONSTRATION OF CORNER FORMULA

Under this condition the corner of the slab acts as a cantilever of uniform strength supporting a load, P, at the extreme corner. (Fig. 7.) At the distance, x, measured diagonally from the corner along the bisector of the right angle of the corner, the bending moment is then Px. This bending moment is resisted by the resisting moment of the section AC, which has a length of $2x$, and a depth of d, the depth of the slab.

The resisting moment is, therefore, equal to

$$\frac{2\,x.d^3S}{12\cdot\frac{d}{2}}=\frac{S\,xd^2}{3}$$

in which S is the unit tensile stress in the top of the slab.

Equating the resisting and bending moments,

$$Px=\frac{S\,xd^2}{3},\text{ or } d=\sqrt{\frac{3\,P}{S}}$$

As tests of the fatigue of concrete have shown that the material will eventually fail under repeated applications of a load which produces a stress equal to approximately 55 per cent of the modulus of rupture. It is not safe to use a value of S in the corner formula in excess of 50 per cent of the modulus of rupture of the material.

Applying the corner formula to the design of slabs finished by the two methods described in this report, it will be observed that Tables 1 and 2 give values of the average modulus of rupture at one year of the normal and Vibrolithic slabs, respectively, for mixes 1:2:3½ and 1:2:4. Similar values at 28 days were given in the earlier report.[5]

The values with the corresponding allowable stresses (one-half of the modulus of rupture) are shown for each mix in the following table.

28-DAY TESTS

Mix	1:2:3½		1:2:4	
Type	Normal	Vibrolithic	Normal	Vibrolithic
	Lbs. per sq. in.	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
Modulus of rupture	586	669	507	579
Allowable stress	293	335	253	289

1-YEAR TESTS

Modulus of rupture	620	782	595	748
Allowable stress	310	391	298	374

Either the 1-year or the 28-day values may be used for substitution in the corner formula to design slabs of equivalent strength, using concrete of the same mix finished by the two methods, but as the pavement is put in service after three or four weeks, it is believed that the 28-day test values are a safer basis for comparison. Using the latter values the design of slabs of uniform depth is illustrated as follows:

SLABS OF SAME MIX AND VARIABLE DEPTH

Assuming a maximum wheel load, P, of 9,000 pounds and a 1:2:3½ mix, the depth of slab required is as follows:

For normal concrete—

$$d=\sqrt{\frac{3\times 9{,}000}{293}}=9.6\text{ inches}$$

For Vibrolithic concrete—

$$d=\sqrt{\frac{3\times 9{,}000}{335}}=9.0\text{ inches}$$

That is, a slab 9 inches thick of the stronger concrete will be equal in flexural strength to a slab of the weaker concrete 9.6 inches thick.

With a 1:2:4 mix and the same wheel load, the equivalent depths will be

For normal concrete,

$$d=\sqrt{\frac{3\times 9{,}000}{253}}=10.3\text{ inches}$$

For Vibrolithic concrete,

$$d=\sqrt{\frac{3\times 9{,}000}{289}}=9.7\text{ inches}$$

[5] Public Roads, vol. 7, No. 2 Apr., 1926, p. 38.

These differences in thickness may be expressed in terms of the quantity of concrete required to lay a lineal yard of 18-foot pavement as follows:

For the 1:2:3½ mix—

	Cubic yards of concrete
Normal slab, 9.6 inches thick, will require	1.60
Vibrolithic slab, 9.0 inches thick, will require	1.50
Difference	.10

As for the 1:2:4 mix—

	Cubic yards of concrete
Normal slab, 10.3 inches thick, will require	1.72
Vibrolithic slab, 9.7 inches thick, will require	1.62
Difference	.10

Thus, with either mix, the reduction in depth permitted by the use of the stronger concrete will make possible a saving of about 0.10 cubic yard of concrete per lineal yard of 18-foot pavement.

THICKENED EDGE PAVEMENTS

By tests and also by theoretical analysis it has been found that a given wheel load will produce a greater stress in a concrete slab if applied at the edge or at a corner, as above, than if the same load is applied in the center of the slab; if the depth of the slab is the same at all points. Such a design is uneconomical because it does not fully employ the strength of the central portions of the slab; and in order that the central portion may carry a stress as great as the edges it has been found that the center depth should be approximately seven-tenths of the edge thickness.

On this basis the uniform-depth design, previously described would be modified as follows:

Assuming—
- 9,000-pound wheel load.
- 1:2:3½ mix, 28 days old.
- Allowable stress in normal concrete=293 pounds per square inch.
- Allowable stress in Vibrolithic concrete=335 pounds per square inch.
- Center thickness=0.7 edge thickness.

Edge thickness by the corner formula:
- 9.6 inches for normal concrete.
- 9.0 inches for Vibrolithic concrete.

Center thickness:
- 6.7 inches for normal concrete.
- 6.3 inches for Vibrolithic concrete.

For an 18-foot pavement with the thickened edge disappearing 2 feet from the edge, the following volumes of concrete are required:

Per lineal yard of pavement:	Cubic yards
Normal	1.17
Vibrolithic	1.10
Difference	.07

SLABS OF SAME CROSS SECTION BUT DIFFERENT MIXES

Using the second method, equal strength in slabs of the two types may be obtained by employing different mixes, and typical data for such alternate designs are supplied directly by this investigation.

The following table gives the average modulus of rupture of the various mixes of the two types at 28 days and 1 year:

Mix	Average modulus of rupture (pounds per square inch)			
	28 days		1 year	
	Normal	Vibrolithic	Normal	Vibrolithic
1:1½:3	624		723	
1:2:3	595		613	
1:2:3½	586	669	620	782
1:2:4	507	579	595	748
1:1½:3½		708		917
1:2:4½		578		673

From these values it will be seen that the following mixes have approximately equal flexural strength:

28 days—1:2:3½ normal and 1:2:4½ Vibrolithic
1 year—1:1½:3 normal and 1:2:4 Vibrolithic

As the equivalent designs, in this case, are of the same thickness there is no difference in the amount of concrete used; but the proportions of cement, sand, and stone are different, and hence there is a difference in cost.

To estimate the difference in cost, the quantities of materials required to produce 1 cubic yard of mixed concrete of the several mixes must be determined and a very satisfactory formula for making this determination has been advanced by Stanton Walker.[6] This formula is based on the assumption known to be practically correct—that the volume of concrete produced by the mixture of cement, and fine and coarse aggregate, and water in given proportions is equal to the combined solid volumes of each of the ingredients.

Thus, if the number of 1-cubic foot sacks of cement entering into a cubic yard of concrete of given mix be designated as N, and if—

x=the ratio of the volume of mixing water to the volume of cement (water-cement ratio);
W_f=the weight of fine aggregate, in pounds, used with one sack of cement;
W_c=the weight of coarse aggregate, in pounds, used with one sack of cement; and—
S_f and S_c=the apparent specific gravities of the fine and coarse aggregates, respectively.

And, finally, the volume of voids in a 1-cubic foot sack of cement being approximately 0.5 cubic foot, Mr. Walker's formula is expressed as follows:

$$0.5N + Nx + \frac{NW_f}{62.5\ S_f} + \frac{NW_c}{62.5\ S_c} = 27 \text{ cubic feet.}$$

Or—

$$N = \frac{27}{0.5 + x + \frac{W_f}{62.5\ S_f} + \frac{W_c}{62.5 S_c}}$$

Using this formula the quantities of materials required to produce 1 cubic yard of mixed concrete of each of the mixes tested in this investigation are as follows:

[6] Walker, Stanton. Estimating quantities of materials for concrete. Bull. No. 1, National Sand and Gravel Association, Washington, D. C.

Quantities of materials required for one cubic yard of mixed concrete

	Mix					
	1:1½:3	1:1½:3½	1:2:3	1:2:3½	1:2:4	1:2:4½
Cement, sacks	6.82	6.31	6.18	5.73	5.35	5.00
Sand, cubic yards	.38	.35	.46	.42	.40	.37
Stone, cubic yards	.76	.82	.69	.74	.79	.83

In calculating the above quantities the average water-cement ratio actually used in the test slabs was employed. These values were:

Mix	Av. $\frac{W}{c}$ (all tests)
1:1½:3	0.72
1:1½:3½	.74
1:2:3	.82
1:2:3½	.86
1:2:4	.90
1:2:4½	.95

Using the above quantities and assumed unit prices of the several materials (average current figures taken from a monthly tabulation by the Engineering News-Record), the cost per cubic yard of concrete of each mix may be calculated as follows:

Cost of materials per cubic yard of concrete

Material	Mix					
	1:1½:3	1:1½:3½	1:2:3	1:2:3½	1:2:4	1:2:4½
Cement, $0.57 per sack	$3.88	$3.60	$3.52	$3.27	$3.05	$2.85
Sand, $1.34 per cubic yard	.51	.47	.62	.56	.54	.50
Stone, $1.83 per cubic yard	1.39	1.50	1.26	1.35	1.45	1.52
Total	5.78	5.57	5.40	5.18	5.04	4.87

Using these costs we may now make an approximate estimate of the comparative costs of the mixes of equivalent strength, above indicated.

Thus—

Mixes of equivalent strength:	Cost
Normal, 1:2:3½	$5.18
Vibrolithic, 1:2:4½	4.87
Difference	.31

Mixes of equivalent strength:	Cost
Normal, 1:1½:3	$5.78
Vibrolithic, 1:2:4	5.04
Difference	.74

Reduced to a square yard basis for an 8-inch pavement this difference becomes $0.069 and $0.164 for the 28-day and 1-year strength values, respectively.

These figures do not include any fabrication costs but, except for the cost of finishing after the concrete is placed, such costs would be the same for the two mixes compared.

To determine exactly what the difference would be it would be necessary to have these costs of the finishing operation by each of the two methods.

Other material prices would also influence the comparison somewhat and the above calculations simply show the approximate cost relation and illustrate one method of making practical use of these test data.

The purpose of the foregoing discussion has been to point out two methods by which the highway engineer may make practical use of data such as are included in this report. The designs employed in the calculations are typical and the unit values are averages, so that the comparisons of the two types of concrete are valid with respect to the particular materials and methods of finishing employed in these tests. It would not be proper to assume, however, that the particular comparisons are valid with respect to the two types of concrete produced under other conditions.

(Continued from p. 178)

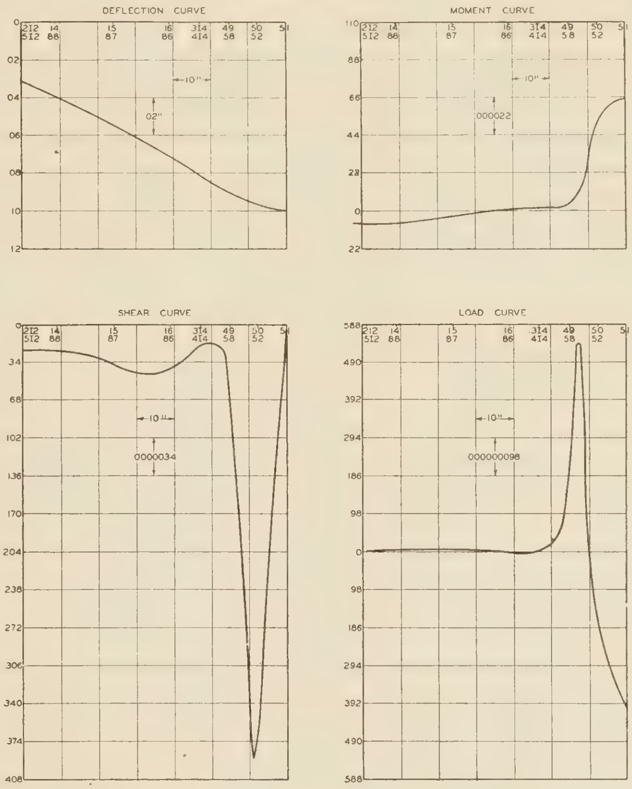


Fig. 48.—Moment, Shear and Load Curves Developed Graphically from Type Deflection Curve on Axis Normal to I Beams for 30,000-pound Load. The Ordinates Shown are Proportional to Moments in Foot-pounds, Shear in Pounds, etc. The Absolute Values are Dependent Upon the Scales Used

By exercising proper care in the selection of the scales, successive curves may be developed from which, by giving due consideration to the variables for any individual slab, ordinates may be scaled which will give the moment, shear, and load at any point on the axes of the slab.

MOTOR VEHICLE REGISTRATIONS FOR THE FIRST SIX MONTHS OF 1927 [1]

State	Motor vehicles registered, individually and commercially owned [2]			Other registered vehicles		Tax-exempt official cars as reported	Total gross receipts	Disposition of gross receipts (approximately allocated when details not reported)					Grand total motor vehicle registration first 6 months of 1926	Increase of 1927 registration over 1926 (6 months) [3]		State
								Collection and administration	For highway purposes			For other purposes				
	Grand total motor cars and trucks, 1927	Passenger automobiles, taxis, and busses	Motor trucks and road tractors	Trailers	Motor cycles				State highways	Local roads	State and county road bonds [4]			Number	Per cent	
Alabama	211,385	184,059	27,326	1,061	287		$2,863,482	$100,819	$1,112,060	$558,379	$1,092,224		197,602	13,783	7.0	Alabama.
Arizona	69,599	59,599	10,000		260	1,000	409,443		409,443				64,165	5,434	8.5	Arizona.
Arkansas	175,709	149,387	26,322	1,430	211	726	3,442,130	37,863	2,244,269	182,433	[5] 977,565		177,235	(1,526)	-.9	Arkansas.
California	1,584,723	1,385,894	198,829	29,570	8,122	9,293	7,862,541	[6] 1,738,841	3,061,850	3,061,850			1,459,570	125,153	8.6	California.
Colorado	234,794	216,031	18,763	77	922		1,436,773	71,839		682,467	682,467		225,810	8,984	4.0	Colorado.
Connecticut	262,035	222,627	39,408	320	2,852	1,755	6,174,232		6,174,232				238,727	23,308	9.8	Connecticut.
Delaware	42,784	34,705	8,079	204	261		775,699		775,699				40,303	2,481	6.1	Delaware.
Florida	373,482	315,009	58,473		1,069	3,387	5,510,715	589,647	3,626,050	1,295,018			[7] 375,700	(2,218)	-.6	Florida.
Georgia	260,079	226,146	33,933		730		3,476,424	89,859	3,386,565				238,618	21,461	9.0	Georgia.
Idaho	89,006	81,161	7,845	133	378	1,174	1,351,986		135,199	1,216,787			84,161	4,845	5.8	Idaho.
Illinois	1,366,060	1,192,286	173,774	3,063	5,318		14,301,788		9,471,998		4,829,790		1,217,265	148,795	12.2	Illinois.
Indiana	745,000	641,881	103,119	5,318	2,689	3,917	5,027,737	190,573	4,837,164				690,704	54,296	7.9	Indiana.
Iowa	660,888	610,415	50,473	150	1,526	2,739	9,977,714	598,663	9,379,051				648,282	12,606	1.9	Iowa.
Kansas	454,685	406,605	48,080		847	2,198	3,405,129	228,967	3,176,162				433,561	21,124	4.9	Kansas.
Kentucky	254,595	227,776	26,819		535	1,679	4,093,126	78,700	3,575,455	438,971			247,104	7,491	3.0	Kentucky.
Louisiana	210,000	178,500	31,500		450		3,954,306		3,954,306				216,500	(6,500)	-3.0	Louisiana.
Maine	141,605	119,261	22,344	755	886	1,117	2,248,145	101,944	1,604,266		479,435	$62,500	128,466	13,139	10.2	Maine.
Maryland	249,883	239,862	10,021	547	2,007	965	2,567,324	256,732	1,797,127			[8] 513,465	227,491	22,392	9.8	Maryland.
Massachusetts	697,401	611,747	85,657	409	5,395	800	10,870,798	1,150,000	8,630,798		1,090,000		627,736	69,668	11.1	Massachusetts.
Michigan	1,041,482	901,651	139,831	12,999	2,806	7,375	16,049,507	258,326	7,550,117	5,992,426	2,248,638		992,178	49,304	5.0	Michigan.
Minnesota	607,725	533,427	74,298	2,838	2,118		9,699,621		7,099,621		[9] 2,600,000		574,356	33,369	5.8	Minnesota.
Mississippi	197,881	178,093	19,788	1,939	57		2,439,514	168,038	200,000	2,071,476			180,630	17,851	9.9	Mississippi.
Missouri	609,849	547,479	62,370		1,466	1,661	7,500,000	1,001,000	2,067,223		4,431,777		583,450	26,399	4.5	Missouri.
Montana	91,701	78,396	13,305		118	248	1,020,380	32,647		915,574		[10] 72,159	92,340	(639)	-.7	Montana.
Nebraska	324,169	297,747	26,422	1,300	679	926	3,318,189	841,729	742,938	1,733,522			329,669	(5,500)	-1.7	Nebraska.
Nevada	22,457	17,858	4,599	71	58	396	208,769	11,491	67,181		130,097		20,527	1,930	9.4	Nevada.
New Hampshire	86,618	76,200	10,418	477	1,059		1,005,769	67,000	938,769				78,979	7,639	9.7	New Hampshire.
New Jersey	639,339	524,550	114,789	1,598	5,504	5,400	11,582,556	368,256	[11] 8,100,000	3,114,300			575,237	64,102	11.1	New Jersey.
New Mexico	50,556	49,315	1,241	109	172	619	495,100	48,707	297,595	148,798			46,571	3,985	8.6	New Mexico.
New York	1,704,987	1,421,562	283,425	5,635	13,095	10,570	28,852,931	1,117,116	20,801,861	6,933,954			1,562,492	142,495	9.1	New York.
North Carolina [1]	418,271	381,338	36,933	1,276	1,020	2,452	5,894,449	397,288	3,890,603		1,606,558		371,353	46,918	12.6	North Carolina.
North Dakota	145,383	133,973	11,410		228		1,413,894	170,000	556,947	556,947		[12] 130,000	144,079	1,304	.9	North Dakota.
Ohio	1,459,815	1,299,606	160,209	10,466	8,284	5,600	8,784,204		4,392,102	4,392,102			1,370,756	89,059	6.5	Ohio.
Oklahoma	459,429	415,929	43,500		835		5,300,000		2,120,000	3,180,000			[13] 441,000	18,429	4.2	Oklahoma.
Oregon	204,895	189,756	[14] 15,139	([14])	1,598	2,068	5,922,214	125,000	4,347,910	1,449,304			195,641	9,254	4.7	Oregon.
Pennsylvania	1,425,424	1,241,247	184,177	3,319	10,526	3,175	22,925,010	614,655	22,024,585			[15] 285,770	1,326,682	98,742	7.4	Pennsylvania.
Rhode Island	103,533	86,503	17,030	42	995	665	1,804,790	100,000	1,704,790				96,652	6,881	7.1	Rhode Island.
South Carolina	174,378	157,087	17,291	1,176	297	3,268	1,976,239	121,450	1,854,789				151,012	23,366	15.5	South Carolina.
South Dakota	152,069	138,855	13,214		159	800	2,346,854	46,937	1,173,427	1,126,490			155,763	(3,694)	-2.4	South Dakota.
Tennessee	265,842	243,443	22,399		643	3,238	3,451,293	41,544	3,409,749				227,775	38,067	16.7	Tennessee.
Texas	950,110	853,881	96,229	6,980	2,335		14,236,439	674,783	8,833,676	4,727,980			904,050	46,060	5.1	Texas.
Utah	84,450	72,927	11,523	149	432	650	603,771	132,000	246,771		225,000		81,830	2,620	3.2	Utah.
Vermont	69,058	63,851	5,207	119	449		1,702,168	44,633	1,657,535				62,899	6,159	9.8	Vermont.
Virginia	299,924	256,358	43,566	285	1,023		4,849,833	67,141	4,782,692				277,125	22,799	8.2	Virginia.
Washington	348,628	298,639	49,989	1,451	1,914	4,434	5,676,974	88,172	4,650,968	937,834			326,500	22,128	6.8	Washington.
West Virginia	205,121	182,816	22,305	292	1,043	2,036	3,525,284	[16] 139,699	2,235,585		1,150,000		183,788	21,333	11.6	West Virginia.
Wisconsin	626,452	548,324	78,128		2,211	391	8,961,029	540,000	5,200,000	3,221,029			590,797	35,655	6.0	Wisconsin.
Wyoming	46,198	40,822	5,376		106	247	478,330		226,551		251,779		44,367	1,831	4.1	Wyoming.
District of Columbia	91,873	80,183	11,690		814	1,915	344,931					[17] 344,931	89,857	2,016	2.2	District of Columbia.
Total	20,991,333	18,414,767	2,576,566	95,558	96,789	88,884	272,119,534	12,452,059	188,525,679	47,937,641	21,795,330	1,408,825	[18] 19,616,755	1,374,578	7.0	Total.

[1] Registration shown for 6 months in all States except North Carolina, which reports full year data, as registration year ends June 30.

[2] These first three columns record the regularly registered motor cars and trucks, excluding where possible the nonresident registrations and reregistations of same vehicle.

[3] Some States report decreases under previous year, indicated by parentheses and in the percentage column by a − sign.

[4] Interest and retirement payments on State road bonds except as indicated.

[5] $82,611 paid on State road bonds, remainder on county road bonds.

[6] Includes undistributed funds.

[7] Formeryl reported as 407,777; the revised figure eliminates nonresident registrations included in 1926 data.

[8] For Baltimore city streets.

[9] Interest and retirement of county bonds.

[10] For auto-theft bureau.

[11] Includes $1,000,000 for bridge and tunnel commission.

[12] For auto-theft law enforcement.

[13] Formerly reported as 490,000; the revised figure eliminates reregistrations included in 1926 published data.

[14] Trailers included with trucks.

[15] For highway motor patrol.

[16] Includes refunds, $23,392.

[17] To U. S. Treasury, from which street appropriations are made for District of Columbia.

[18] Revised total due to revisions in Florida and Oklahoma as noted.

U. S. GOVERNMENT PRINTING OFFICE: 1927

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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 REPORTS

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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*136D. Highway Bonds. 20c.
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Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
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

SEPTEMBER 30, 1927

STATES	FISCAL YEARS 1917-1927			FISCAL YEAR 1928									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1927			PROJECTS COMPLETED SINCE JUNE 30, 1927			*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	$ 20,061,371.68	$ 9,615,099.94	1,400.2	$ 84,878.41	$ 40,723.98	0.2	$ 7,048,755.05	$ 3,368,691.51	409.8	$ 39,289.25	$ 19,644.62	0.2	$ 2,852,514.95	Alabama
Arizona	11,809,950.70	6,447,169.27	800.8	267,759.35	192,918.61	14.1	1,448,605.72	1,088,686.44	67.8	34,528.88	17,264.44		2,928,204.24	Arizona
Arkansas	22,337,014.63	9,525,192.75	1,560.6	52,245.29	26,122.64	0.1	3,613,444.42	1,880,893.63	223.9	717,686.26	158,899.73	7.3	1,292,591.25	Arkansas
California	35,128,269.04	16,967,026.82	1,306.3	826,669.60	429,604.25	29.8	7,526,051.48	3,463,363.06	150.1	31,568.35	18,941.01	0.1	3,677,316.86	California
Colorado	15,497,121.91	7,934,298.91	829.0	91,935.74	48,486.64	0.9	6,387,143.17	3,121,842.16	280.3	35,740.10	20,057.34	2.9	2,577,646.95	Colorado
Connecticut	6,397,392.29	2,444,000.54	137.3	961,143.03	203,730.00	13.6	5,185,358.30	1,522,599.35	60.6	968,919.99	251,363.49	13.8	384,672.62	Connecticut
Delaware	6,237,026.55	2,345,572.42	159.5	263,662.17	120,820.22	14.6	539,578.36	188,470.46	15.7	458,820.37	176,900.72	15.4	7,919.18	Delaware
Florida	7,476,856.31	3,627,912.60	245.1	1,517,818.43	734,206.79	32.7	8,217,755.11	3,507,767.54	176.2	777,845.42	269,730.00	18.0	844,788.07	Florida
Georgia	31,951,436.50	15,101,232.40	2,173.6	2,340,425.97	1,113,809.87	86.1	7,407,445.95	3,668,582.92	286.5	379,747.46	188,028.25	0.1	339,508.56	Georgia
Idaho	13,225,515.45	7,075,527.16	835.5	149,006.57	87,998.40	24.2	2,808,373.89	1,743,761.70	187.9	666,191.56	397,052.97	36.3	190,479.77	Idaho
Illinois	48,538,982.16	22,781,516.60	1,530.8				14,556,820.36	7,016,546.35	505.6	2,527,747.47	1,258,103.41	86.7	1,930,460.64	Illinois
Indiana	23,372,717.74	11,238,568.20	732.5	406,186.56	191,269.03	13.3	16,868,915.82	8,075,101.26	507.8	1,136,510.75	373,205.37	43.1	252,983.14	Indiana
Iowa	34,306,138.86	14,395,603.75	2,484.4	709,235.61	344,072.82	53.8	15,357,626.83	6,680,555.95	549.3	539,012.31	98,386.66	15.2	11,942.82	Iowa
Kansas	37,442,061.61	14,730,829.48	1,495.2	671,104.37	299,030.45	36.7	14,839,354.09	5,871,973.53	762.0	1,056,933.04	381,057.44	99.1	250,052.10	Kansas
Kentucky	23,216,600.53	9,510,694.75	874.9	430,346.22	201,333.62	20.6	10,020,663.35	4,730,589.55	434.1	341,085.45	170,542.71	16.6	17,595.37	Kentucky
Louisiana	15,877,552.20	7,093,892.21	1,178.7	931,554.63	442,316.21	37.9	3,421,391.18	1,523,675.46	95.6	1,208,085.09	604,042.54	13.1	621,789.58	Louisiana
Maine	10,564,800.06	4,858,452.67	357.6	304,561.61	146,303.87	17.3	2,764,187.82	918,131.24	64.0	391,447.59	195,499.27	19.9	1,027,234.95	Maine
Maryland	11,790,203.93	5,524,938.27	477.8				1,236,282.89	569,085.64	53.4	1,029,338.00	457,800.00	51.2	8,352.09	Maryland
Massachusetts	20,670,246.02	7,425,928.15	410.4	494,913.79	93,825.00	6.3	6,427,405.40	1,738,239.63	105.7	109,164.82	18,675.00	1.3	1,921,158.22	Massachusetts
Michigan	31,977,248.37	14,328,484.99	1,084.2	81,313.81	37,302.76	2.0	14,112,628.30	6,399,277.08	408.7	2,168,320.00	880,175.00	43.2	911,816.17	Michigan
Minnesota	45,099,648.47	19,046,145.67	3,643.5				7,533,620.17	2,158,898.90	392.0	443,394.96	6,000.00	13.5	501,476.43	Minnesota
Mississippi	18,331,230.75	9,004,294.62	1,314.1	805,968.50	402,478.38	45.3	6,414,826.62	3,098,087.90	360.2	569,535.25	284,767.62	23.4	646,268.48	Mississippi
Missouri	42,389,290.41	19,681,026.48	1,944.8	1,822,276.10	730,654.06	57.1	8,861,405.29	3,840,278.04	274.2	537,680.92	267,955.91	30.4	671,696.51	Missouri
Montana	12,854,995.72	7,287,288.69	1,151.6	206,458.10	152,882.59	25.3	3,202,940.76	2,277,574.70	243.5	1,837,105.41	1,069,313.87	193.2	4,189,324.15	Montana
Nebraska	16,157,040.25	7,739,386.39	2,246.6	2,101,062.18	1,007,922.26	198.8	12,853,274.51	6,332,966.99	1,298.5	993,973.25	490,350.80	68.2	649,746.56	Nebraska
Nevada	10,421,349.31	7,589,168.68	853.6	94,058.56	82,449.77	27.4	1,809,604.95	1,571,810.67	193.7	82,385.52	72,268.56	9.4	428,027.32	Nevada
New Hampshire	5,868,897.76	2,778,928.05	264.8	89,474.44	44,163.01	3.7	1,454,099.40	631,427.85	41.1	174,090.36	77,846.01	6.3	2,752.08	New Hampshire
New Jersey	22,228,240.08	7,495,364.48	316.3	1,359,957.39	277,170.00	18.5	6,962,829.39	1,601,398.56	104.7	94,656.00	28,095.00	1.9	2.96	New Jersey
New Mexico	13,336,250.94	7,937,598.06	1,505.2	160,059.48	100,334.00	18.3	3,217,758.11	2,500,113.06	234.7				1,621,103.88	New Mexico
New York	54,183,085.44	21,693,955.65	1,439.3	1,782,436.58	545,424.84	37.5	40,165,536.00	10,090,878.95	622.0	9,351,100.00	1,817,612.50	117.3	3,532,540.06	New York
North Carolina	35,295,849.21	14,518,903.16	1,460.1	747,062.66	349,898.15	28.6	3,144,127.66	1,482,854.57	80.4	777,080.50	363,253.75	26.3	715,652.37	North Carolina
North Dakota	15,881,558.55	7,746,293.68	2,715.6	1,243,277.39	730,757.44	49.5	5,434,696.01	2,814,778.56	894.5	1,176,170.82	516,579.76	210.6	135,200.56	North Dakota
Ohio	52,621,391.49	19,331,376.76	1,515.0	541,204.15	251,965.07	16.9	11,459,970.84	4,410,653.90	325.6	4,571,142.00	1,751,215.00	84.1	2,748,794.27	Ohio
Oklahoma	30,381,957.08	14,117,589.21	1,268.1	789,742.25	379,754.99	9.6	4,237,934.32	1,803,363.89	301.9	1,669,956.28	818,709.28	86.6	692,260.63	Oklahoma
Oregon	19,583,584.76	10,041,452.94	1,055.0	161,863.43	92,383.50	0.2	2,452,045.21	1,277,261.67	72.8	226,673.92	126,976.58	6.1	523,474.31	Oregon
Pennsylvania	77,726,174.22	26 317,620.32	1,534.3	869,406.01	282,528.10	22.9	18,738,648.86	5,695,826.02	370.7	2,924,606.80	951,388.84	43.8	1,427,152.72	Pennsylvania
Rhode Island	5,233,413.38	1,998,479.06	116.0				1,815,941.88	480,563.74	31.1	427,199.34	128,638.67	7.6	425,512.53	Rhode Island
South Carolina	17,002,039.93	7,526,988.80	1,568.4	1,420,754.23	688,413.55	39.3	7,693,268.91	2,407,723.43	257.6	59,479.64	9,000.00		224,386.22	South Carolina
South Dakota	19,262,063.24	9,507,525.54	2.502.9	109,739.28	57,825.99	29.1	4,443,470.27	2,420,862.36	747.5	494,552.37	272,003.80	68.1	128,636.31	South Dakota
Tennessee	24,283,035.03	11,551,457.55	868.7	1,449,112.56	632,743.67	24.6	7,294,823.16	3,243,456.92	217.1	1,109,002.91	466,756.41	41.9	1,000,942.45	Tennessee
Texas	78,190,246.37	31,566,960.45	5,466.4	3,543,794.64	1,555,087.85	124.6	14,425,659.14	6,515,476.56	497.3	2,928,218.13	1,216,404.43	100.2	4,249,773.71	Texas
Utah	9,154,377.33	5,767,079.95	628.9	393,000.76	298,136.85	35.9	2,692,200.63	2,005,007.34	185.8	610,879.46	422,096.65	37.0	173,364.21	Utah
Vermont	5,037,118.23	2,348,856.01	152.7	91,777.53	40,673.05	1.3	3,426,927.10	1,243,622.61	70.3				980.33	Vermont
Virginia	26,844,025.24	12,537,143.25	1,168.9	275,796.37	117,551.38	8.2	5,261,354.54	2,121,948.18	113.1	336,186.92	150,738.21	9.1	16,846.98	Virginia
Washington	18,184,505.97	8,246,551.95	711.1	847,151.85	415,736.32	5.3	2,918,690.33	1,330,600.00	66.5	1,243,406.64	508,000.00	48.3	776,419.73	Washington
West Virginia	10,424,847.32	4,573,748.01	419.4				7,123,946.40	2,929,734.05	238.2	1,500,484.17	617,475.22	48.8	25,139.72	West Virginia
Wisconsin	27,891,502.16	11,847,858.90	1,729.5	1,650,332.14	835,073.17	94.4	9,534,949.13	4,338,085.18	336.2	48,178.06	24,000.00	3.5	2,264,252.75	Wisconsin
Wyoming	12,650,712.15	7,139,267.05	1,315.9	218,889.07	143,801.81	24.2	2,740,967.37	1,753,760.77	190.4	197,169.20	126,582.59	39.6	337,230.78	Wyoming
Hawaii	343,664.15	97,440.00	6.5				1,786,323.89	562,362.64	29.7				805,975.36	Hawaii
TOTALS	1,154,740,601.48	510,007,691.24	60,957.6	33,359,416.81	14,969,684.96	1,350.7	358,889,628.44	150,019,212.47	14,136.3	49,002,290.99	18,539,399.43	1,808.7	50,964,011.90	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 101,878,159.50 Federal aid $ 44,022,355.63 Miles 4,214.9