PUBLIC ROADS

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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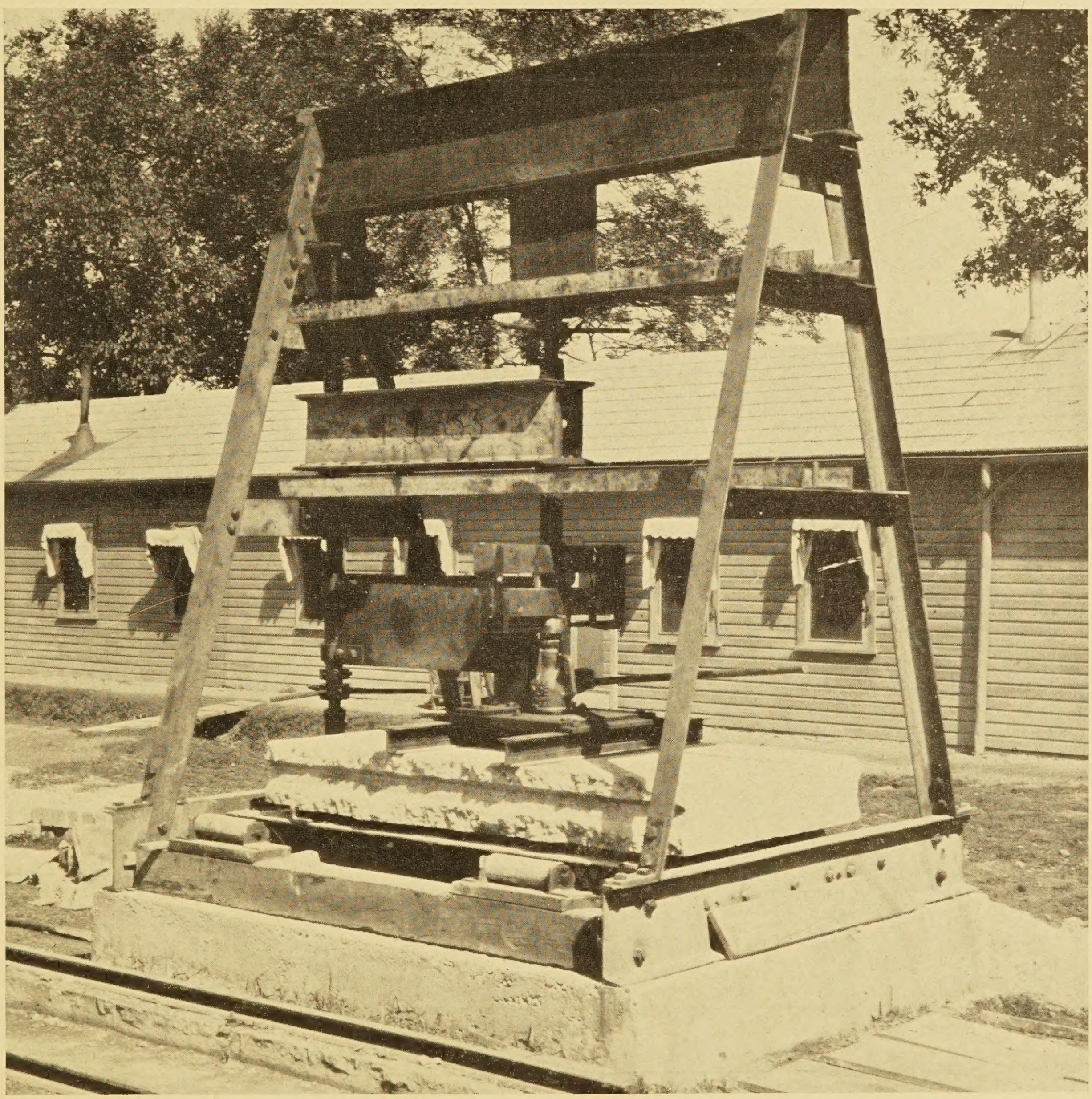

MACHINE FOR DETERMINING FLEXUAL STRENGTH OF CONCRETE SLABS

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THE EFFECT OF MATERIALS AND METHODS OF PLACING ON THE STRENGTH AND OTHER PROPERTIES OF CONCRETE BRIDGE FLOOR SLABS

REPORT OF A COOPERATIVE INVESTIGATION CONDUCTED BY THE PORT OF NEW YORK AUTHORITY AND THE UNITED STATES BUREAU OF PUBLIC ROADS

By L. W. TELLER, Senior Engineer of Tests, and G. W. DAVIS, Associate Engineer of Tests, United States Bureau of Public Roads

IN THE field of highway bridge engineering one of the most important developments of the times is the trend toward increased span lengths. This development has been made possible, in a large measure, by new materials, new methods of construction, and new knowledge which have been made available to the bridge engineer within the past few years. While the general tendency toward greater span lengths has been evident in the design of many types of bridges, it has been particularly noticeable in that of suspension bridges. A span of 1,800 feet is a thing of the past; one of 3,500 feet has recently been opened to traffic, and another of much greater length is already planned.

With such great span lengths the possibilities for economy through a reduction in the dead load caused by the weight of the floor system are of such importance as to merit the designer's most serious consideration. Nearly three years ago, when the design for the giant bridge over the Hudson River between Manhattan at One hundred and seventy-eighth Street and Fort Lee, N. J., was being developed, this thought led the Port of New York Authority to request the Bureau of Public Roads to cooperate in the conduct of a rather extensive series of tests, the primary object of which was the development of information which would permit a comparison to be made of the relative strengths of concrete floor slabs made of several materials and placed by each of several methods.

The tests requested were begun during the summer of 1928 and have been completed only quite recently, because of the time required to make certain extended durability tests.

SCOPE OF TESTS OUTLINED

The materials which it was desired to compare were:

1. Coarse aggregates—
   - *a.* Haydite (an artificial lightweight aggregate).
   - *b.* Rounded siliceous gravel.
   - *c.* Crushed limestone.
2. Fine aggregates—
   - *a.* Haydite.
   - *b.* Natural quartz sand.

The methods of consolidation used in the placing of the concrete were as follows:

1. Hand placing.
2. Vibrolithic process.
3. Vibration with an electric vibrator.
4. Vibration with an air hammer.

The detailed descriptions of the materials and of the methods of consolidation are given later in the report.

To provide satisfactory specimens for the tests it was necessary to construct a series of slabs sufficient in size to permit all operations to be carried out as they would be in actual construction. Since, in some cases, vibration was to be used for the consolidation of the concrete, it was necessary that the method of supporting the slabs during construction be comparable to that which would normally be used in the design of such floor systems.

The series consisted of 11 slabs, in duplicate, or a total of 22 slabs each 22 feet long, 6 feet wide, and 8 inches thick. In their construction the various materials and methods were combined as shown in Table 1. It will be seen from this table that in placing the concrete containing the Haydite aggregates the air hammer was not used. This omission was made because it was believed that a satisfactory comparison between methods could be made on the other two materials and that sufficient data concerning Haydite would result from the slabs made by the other three methods.

TABLE 1.—*List of test sections*

Slab numbers	Method of placing	Aggregates
13 and 24	Hand placed	Crushed limestone and natural sand.
12 and 23	do	Gravel and natural sand.
3 and 14	do	Fine and coarse Haydite.
5 and 16	Vibrolithic process [1]	Crushed limestone and natural sand.
4 and 15	do	Gravel and natural sand.
6 and 17	do	Fine and coarse Haydite.
8 and 19	Vibrated with electric tamper	Crushed limestone and natural sand.
7 and 18	do	Gravel and natural sand.
9 and 25	do	Fine and coarse Haydite.
11 and 22	Vibrated with air hammer	Crushed limestone and natural sand.
10 and 21	do	Gravel and natural sand.

[1] To each of the 6 slabs placed by the Vibrolithic process there was added trap rock in the proportion of 50 pounds per square yard.

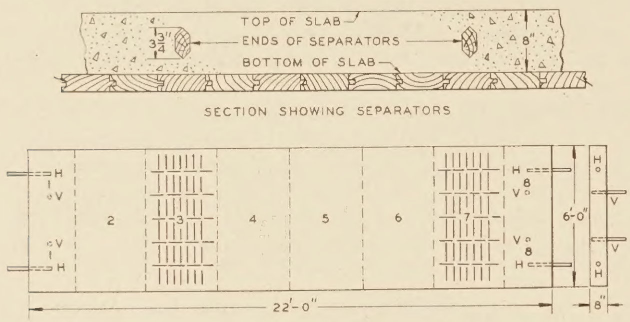
The concrete in each of these slabs was subjected to the following tests:

1. Flexural strength—
   - *a.* Flexural strength of 36 by 72 inch slabs, 8 inches thick.
   - *b.* Flexural strength of 8 by 8 by 36 inch sawed beams.
2. Compressive strength—
   - *a.* Compressive strength of 6 by 8 inch drilled cores.
   - *b.* Compressive strength of 6 by 12 inch control cylinders.
3. Modulus of elasticity—
   Modulus of elasticity of 6 by 8 inch drilled cores.
4. Bond strength of embedded steel—
   - *a.* Vertically placed bars.
   - *b.* Horizontally placed bars.
5. Stress and deflection measurements in reinforced slabs, 36 by 72 inches in area.
6. Absorption tests.
7. Wear tests, to determine relative surface hardness of the concrete.
8. Tests of resistance to alternate freezing and thawing.

GENERAL PROCEDURE DESCRIBED

In order that the effect of construction methods might be the same on the test slabs as on actual bridge floor slabs, the test slabs were constructed on stringers which consisted of 18-inch, 54.5-pound I beams, 22 feet long, placed 5 feet 4 inches on centers and supported at either end by short concrete pedestals.

Although each of the large slabs was constructed as a unit, it was divided in the forms into sections of a size suitable for testing by wooden dividing strips or partitions about half the depth of the forms, or 3¾ inches in width. These dividing strips, 2 inches thick with chamfered edges, were fastened in the forms in a very dry condition. On placing, the wet concrete caused these strips to swell sufficiently to crack the large slabs into smaller sections. Details of the method of dividing the large slab are shown in Figures 1 and 2.



1 & 8 — 2 BY 3 FOOT SLABS FOR BOND TESTS
H = HORIZONTAL BARS, V = VERTICAL BARS
3 & 6 — 3 BY 6 FOOT REINFORCED SLABS
2 4 5 & 7 — 3 BY 6 FOOT UNREINFORCED SLABS
------- SEPARATORS

FIGURE 1.—DETAILS OF TEST SLABS

MATERIALS USED

The materials used were obtained from the same sources as the materials contemplated for use on the bridge. Their physical characteristics are given below.

Cement.—The cement used was a reputable brand of Portland cement with the following characteristics. The values given are the averages of tests of five samples.

Fineness, percentage retained on 200-mesh sieve	19.6
Soundness	Satisfactory.
Time of set (Gilmore needle):	
Initial	3 hours, 15 minutes.
Final	7 hours, 0 minutes.
Normal consistency, per cent	21.6
Tensile strength (pounds per square inch—1 : 3 Ottawa sand mortar):	
7 days	253 pounds.
28 days	365 pounds.

Fine aggregates.—The sand used as a fine aggregate in these tests came from Suffolk County, N. Y. It consists essentially of angular grains of quartz, containing a small amount of mica and feldspar. Physical tests gave the following results:

Sieve analysis:	
Total retained on—	
No. 4 sieve .... per cent	2
No. 8 sieve .... do	13
No. 14 sieve .... do	24
No. 28 sieve .... do	49
No. 48 sieve .... do	87
No. 100 sieve .... do	98
Silt and clay .... do	0.4
Apparent specific gravity	2.67
Weight in pounds per cubic foot (dry rodded)	105
Organic matter (color test)	Satisfactory.
Strength ratio—	
7 days .... per cent	116
28 days .... do	106
Fineness modulus	2.73

Haydite is a manufactured aggregate consisting of fused clay or shale. It was obtained from Jackson County, Mo. Haydite

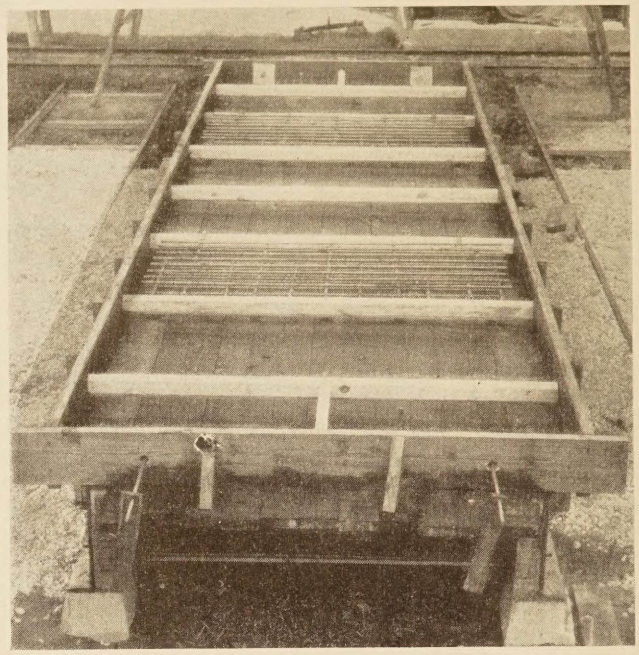


FIGURE 2.—FORM READY FOR CONSTRUCTION OF LARGE SLAB. NOTE SEPARATORS, STEEL MATS, AND BARS FOR BOND TESTS

No. 6, which was used as a fine aggregate, gave the following test results:

Sieve analysis:	
Total retained on—	
No. 4 sieve .... per cent	0
No. 8 sieve .... do	5
No. 16 sieve .... do	37
No. 30 sieve .... do	60
No. 50 sieve .... do	74
No. 100 sieve .... do	83
Apparent specific gravity	1.61
Weight in pounds per cubic foot (dry rodded)	68.75
Percentage absorption	14.7
Fineness modulus	2.59

Coarse aggregates.—The gravel, which was obtained from Suffolk County, N. Y., consisted essentially of rounded fragments of sandstone and quartz. Prior to the mechanical analysis the gravel as received was separated into four sizes and recombined as follows:

Over 1¼ inches .... per cent	23
¾ to 1¼ inches .... do	37
¼ to ¾ inch .... do	32
Less than ¼ inch .... do	8
Mechanical analysis—	
Total retained on—	
1½-inch screen .... do	9
¾-inch screen .... do	52
⅜-inch screen .... do	78
No. 4 sieve .... do	95
No. 8 sieve .... do	99
Apparent specific gravity	2.61
Weight in pounds per cubic foot (dry rodded)	112
Percentage absorption	0.28
Fineness modulus	7.33

Following are the results of physical tests on the crushed stone, a siliceous limestone from Rockland County, N. Y.

Mechanical analysis:	
Total retained on—	
1½-inch screen .... per cent	3
¾-inch screen .... do	52
⅜-inch screen .... do	88
No. 4 sieve .... do	98
No. 8 sieve .... do	99
No. 16 sieve .... do	100
Apparent specific gravity	2.82
Weight in pounds per cubic foot (dry rodded)	106
Percentage absorption	0.05
Percentage wear	4.5
Fineness modulus	7.40

Following are the results of physical tests on the ¾-inch Haydite which was used as coarse aggregate:

Mechanical analysis:

Total retained on—	
1½-inch screen .... per cent	0
¾-inch screen .... do	1
⅜-inch screen .... do	34
No. 4 sieve .... do	85
No. 8 sieve .... do	96
No. 16 sieve .... do	100
Apparent specific gravity	1. 31
Weight in pounds per cubic foot (dry rodded)	51. 75
Percentage absorption	7. 4
Fineness modulus	6. 16

The trap rock which was used in the Vibrolithic concrete was obtained from Rockland County, N. Y. Following are its characteristics:

Size: 1 to 2 inches.	
Apparent specific gravity	2. 83
Percentage absorption	0. 18
Description: Gabbroitic diabase.	

DESIGN OF CONCRETE MIXES

The mixtures used were modifications of experimental mixes for the particular aggregates used, as developed in the testing laboratory of the Port of New York Authority. In both the gravel and the limestone mixes the grading of the aggregates was in accordance with specifications of the Port of New York Authority, as were the fineness moduli of the combined aggregates. For these two aggregates the real mix was identical, although the percentage of fine to coarse aggregate was varied slightly to give the desired workability. In the case of the Haydite aggregates the mix was fixed as a 1:1½:3 loose volume mix, with 3 pounds of Celite added to each sack of cement, this having been found the most satisfactory of several experimental mixes by the laboratory of the Port of New York Authority. The high absorption of Haydite made it necessary to use the material in a saturated condition in order to control the water cement ratio. Both the fine and the coarse Haydite, packed in burlap sacks, were completely submerged in water for from 16 to 20 hours. The sacks of material were then removed from the water and allowed to drain for one hour before mixing was begun.

Surface moisture determinations were made for both Haydite aggregates and for the sand just prior to their use, and an allowance for it was made in calculating the water-cement ratios. As both the gravel and the stone aggregates were sun-dried, sacked, and stored under cover, no measurable surface moisture was present. All mixtures were made as dry as compatible with the methods of placing used, the consistency of all mixes being limited to a slump of 1½ inches. Table 2 gives the complete mixing data. The batch quantities shown are the actual weights of the materials used, including surface moisture.

TABLE 2.—*Concrete design data for bridge floor slabs*

Slab No.	Surface moisture: Fine aggregate	Surface moisture: Coarse aggregate	Batch quantities: Water	Batch quantities: Cement	Batch quantities: Fine aggregate	Batch quantities: Coarse aggregate	Water-cement ratio	Real mix	Nominal mix	Cement factor	Percentage fine aggregate	Percentage coarse aggregate	Fineness modulus of mixed aggregate	Type of aggregate and method of placing
	Per cent	*Per cent*	*Pounds*	*Pounds*	*Pounds*	*Pounds*				*Bags per cubic yard*				
3	13.9	4.2	⅛–31.5 ⅞–34.5	94	100.1	150.6	0.83 .88	1:3.88	1:1.25:2.79	7.96	30.9	69.1	5.06	Haydite, hand placed.
4	1.8	0	49.4	94	225.0	420.0	.86	1:4.79	1:2.10:3.75	5.03	35.9	64.1	5.68	Gravel, vibrolithic.
5	1.9	0	51.7	94	243.0	383.0	.90	1:4.65	1:2.27:3.61	5.27	38.6	61.4	5.60	Stone, vibrolithic.
6	14.8	2.9	28.5	94	97.1	151.1	.76	1:3.88	1:1.20:2.84	7.76	29.7	70.3	5.10	Haydite, vibrolithic.
7	1.8	0	49.4	94	230.5	430.5	.86	1:4.92	1:2.16:3.84	5.26	36.0	64.0	5.67	Gravel, electric tamper.
8	1.4	0	53.2	94	258.0	402.0	.91	1:4.91	1:2.42:3.79	5.39	39.0	61.0	5.58	Stone, electric tamper.
9	14.3	4.2	½–28.5 ½–28.0	94	103.4	152.3	.80 .79	1:3.95	1:1.29:2.82	7.94	31.4	68.6	5.04	Haydite, electric tamper.
10	2.2	0	48.2	94	231.5	430.5	.85	1:4.92	1:2.16:3.84	5.18	36.0	64.0	5.67	Gravel, air hammer.
11	2.8	0	49.4	94	262.0	402.0	.91	1:4.91	1:2.43:3.79	5.28	39.1	60.9	5.57	Stone, air hammer.
12	2.5	0	47.6	94	232.6	430.5	.86	1:4.92	1:2.16:3.84	5.02	36.0	64.0	5.67	Gravel, hand placed.
13	2.9	0	49.1	94	262.8	402.8	.91	1:4.92	1:2.43:3.80	5.20	39.0	61.0	5.58	Stone, hand placed.
14	13.3	4.7	28.0	94	102.5	149.2	.78	1:3.88	1:1.29:2.75	8.18	31.9	68.1	5.02	Haydite, hand placed.
15	1.8	0	49.4	94	230.5	430.5	.86	1:4.92	1:2.16:3.84	4.98	36.0	64.0	5.67	Gravel, vibrolithic.
16	2.5	0	50.2	94	261.8	402.8	.91	1:4.92	1:2.43:3.80	5.00	39.0	61.0	5.58	Stone, vibrolithic.
17	13.9	4.5	⅜–28.0 ⅛–27.5 ½–27.0	94	104.8	150.7	.79 .78 .78	1:3.93	1:1.31:2.78	7.72	32.0	68.0	5.02	Haydite, vibrolithic.
18	1.8	0	49.4	94	230.5	430.5	.86	1:4.92	1:2.16:3.84	5.18	36.0	64.0	5.67	Gravel, electric tamper.
19	1.7	0	52.4	94	259.5	402.8	.91	1:4.92	1:2.43:3.80	5.43	39.0	61.0	5.58	Stone, electric tamper.
21	1.8	0	49.4	94	230.5	430.5	.86	1:4.92	1:2.16:3.84	5.11	36.0	64.0	5.67	Gravel, air hammer.
22	2.6	0	50.0	94	262.0	402.8	.91	1:4.92	1:2.43:3.80	5.29	39.0	61.0	5.58	Stone, air hammer.
23	2.6	0	47.2	94	232.9	430.5	.85	1:4.92	1:2.16:3.84	5.15	36.0	64.0	5.67	Gravel, hand placed.
24	2.5	0	50.2	94	261.8	402.8	.91	1:4.92	1:2.43:3.80	5.11	39.0	61.0	5.58	Stone, hand placed.
25	9.4	6.5	27.0	94	93.8	149.6	.73	1:3.78	1:1.24:2.70	8.43	31.5	68.5	5.04	Haydite, electric tamper.

SEPARATE AGGREGATES

	Fineness modulus	Weight per cubic foot	Specific gravity	Percentage voids (calculated)	Absorption
		Pounds			*Per cent*
Sand	2.73	105.00	2.67	37.0	1.00
Gravel	7.33	112.00	2.61	31.2	.28
Limestone	7.40	106.00	2.82	39.8	.05
Trap			2.83		.18
Haydite No. 6	2.59	68.75	1.61	31.6	14.70
Haydite, coarse	6.16	51.75	1.31	36.7	7.40

MIXED AGGREGATES

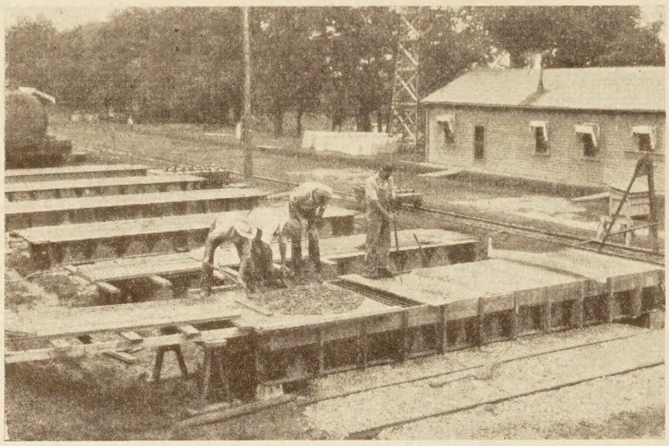
	Weight per cubic foot	Shrinkage factor
	Pounds	
Sand and gravel	133.9	0.82
Sand and limestone	134.1	.79
Haydite	69.6	.96

METHODS OF CONSTRUCTION

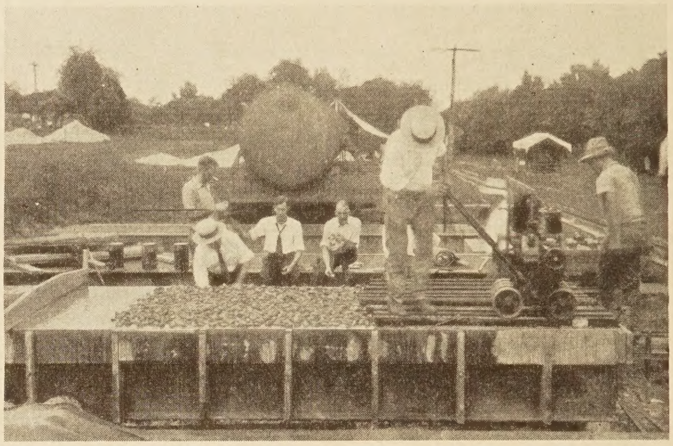
In the construction of the test slabs six forms were used. These forms, constructed of 2-inch tongue-and-groove planking, were set up with 8-foot intervals between them. Down the center of these 8-foot spaces narrow-gage tracks were laid running parallel to the long axes of the slabs. Perpendicular to these tracks and passing along one end of the forms, there was laid another narrow-gage track which led to a field testing machine designed to make cross-bending tests on the 3 by 6 foot sections. Narrow-gage cars with platform decks fitted with rollers were used to move the test specimens.

All aggregates were stored and all mixing was done on a terrace or bench about 4 feet above the level of the forms.

The mixing was done in a 10-cubic-foot drum-type mixer electrically driven at the rate of 20 r. p. m. All 2,200 vibrations per minute for a period of three to four minutes for each 6-foot section. As the forms were filled the platforms were moved forward in steps of approximately 3 feet and the concrete vibrated anew, five positions of the platforms covering the entire 22-foot slab. The irregular mortar surface left on removal of the platforms was smoothed off with a wooden strike board and the surface finished by belting. These slabs were also covered with wet burlap.

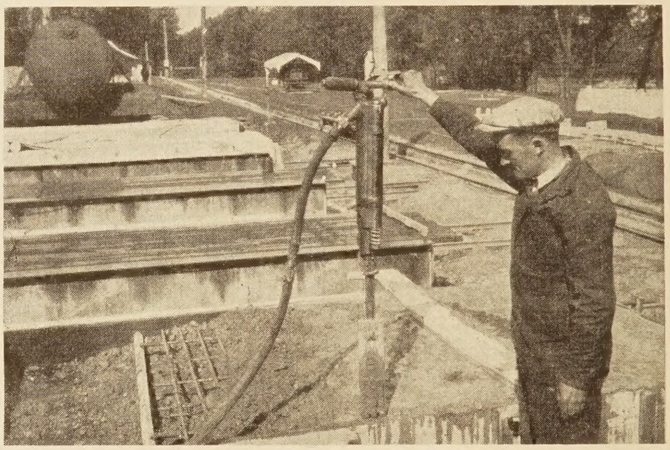


HAND PLACING



VIBROLITHIC PROCESS

ELECTRIC TAMPER



AIR HAMMER

FIGURE 3.—THE FOUR METHODS USED FOR CONSOLIDATING THE CONCRETE

batches were mixed for two minutes and then discharged into concrete buggies. The buggies were wheeled to a platform resting on top of the form, where they were dumped. The concrete was shoveled from this platform into place in the slab forms and compacted and finished by one of the following methods.

Hand placing.—In the case of the hand-placed slabs the concrete was well spaded along the edges of the forms, tamped across once with an 8 by 2 inch steel-edged strike board, and then struck off. The slab was then belted with an 8-inch rubber belt and covered with wet burlap as soon as the surface would allow.

Vibrolithic process.—The concrete in these slabs was placed and struck off in the same manner as in the case of the hand-placed slabs. As soon as the surface of the slab had been struck off it was covered uniformly with trap rock, 1 to 2 inches in size, in the proportion of 50 pounds to the square yard of surface. On this stone special slatted platforms, 6 feet long by 14 inches wide, were placed. Over these platforms the patented vibrator was rolled and the concrete subjected to about

Two types of platform were used in these tests. The first series of Vibrolithic slabs was finished with the standard platform which has ½-inch spaces between the cleats. On the platforms used in the second series the cleat spacing was reduced to one-fourth inch in an attempt to reduce the amount of mortar brought to the surface. This modification apparently reduced somewhat the thickness of the mortar cover.

The electric tamper.—The placing of concrete by this method followed the procedure used in the case of the hand-placed slabs up to and including the shoveling of the concrete from the dumping platform into the forms. The concrete once in place was vibrated by means of a patented electric tamper which causes a vibration of relatively small displacement but of high frequency, quite different from the direct but slower blows of the air hammer. The tamper consists essentially of an unbalanced weight on the shaft of a rotor revolved by an electric motor. When this rotor is revolved at its normal speed of 3,600 r. p. m. the unbalanced weight sets up vibrations in the entire device. These vibra-

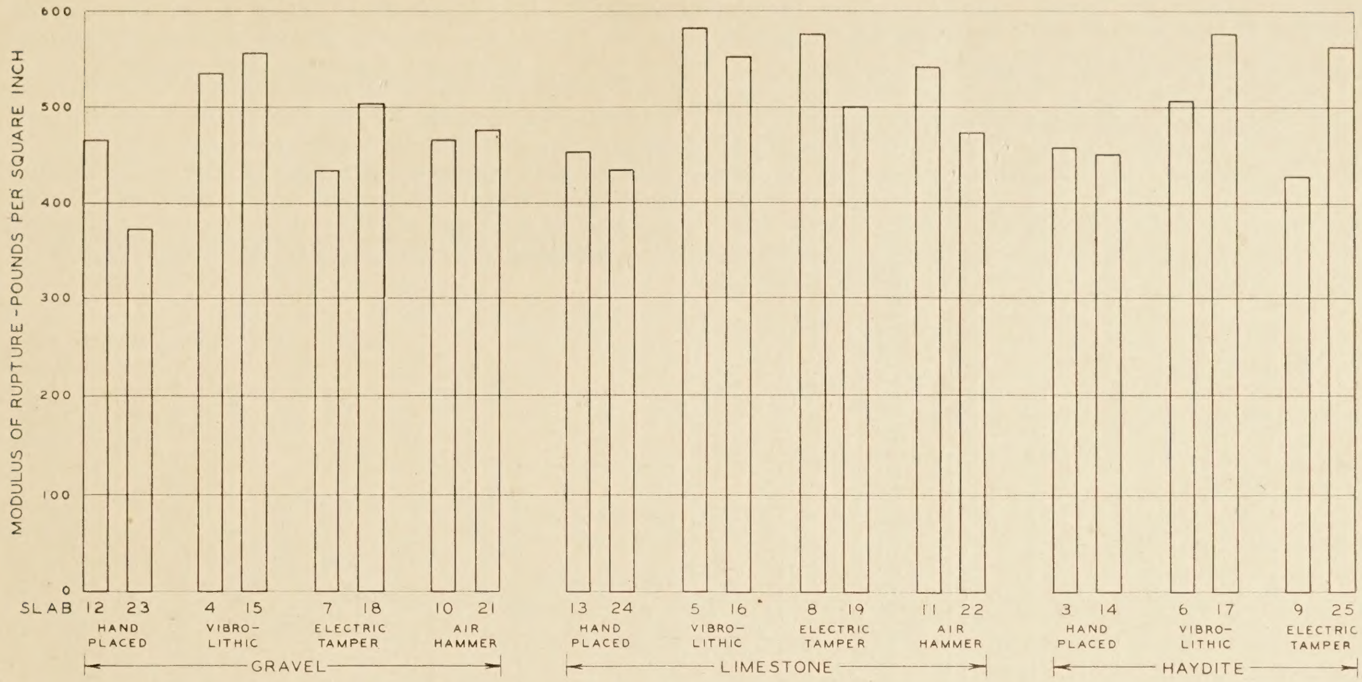


Figure 4.—Values of Modulus of Rupture of 36 by 8 by 72 Inch Slabs Tested at 22 Days. Each Value Is the Average of Four Tests

tions are transmitted to the form and to the concrete by means of a wooden tamping bar bolted to the motor frame. The vibrations are reduced before reaching the operator through spring connections between the motor and the operating handles. The total length of time that each 22-foot slab was vibrated was about 20 minutes. The slabs were finished with the wooden strike board and belted, and were covered with wet burlap immediately after the initial set.

The air hammer.—The procedure in placing this type of slab was the same as for the electric tamper, except that the concrete was compacted by vibrating the steel I-beam stringers, and through them the forms and concrete, by means of an air hammer. The hammer used was a Cleveland, type C 6 hammer, weighing 85 pounds and having a rated delivery of from 1,200 to 1,250 blows of 27 ± foot-pounds per minute when operated under 65 pounds air pressure. A wooden tamping bar 2 by 4 inches in cross section bolted to a section of a steel cutting tool was inserted in the hammer. Each I-beam was vibrated for approximately one-half a minute for each one-bag batch, the total time of vibration for the entire slab being about 20 minutes, or the equivalent of the period of vibration used in the Vibrolithic and electric tamper placements.

Figure 3 shows the equipment used in each of the four methods of placing.

Curing and storing.—At the end of 24 hours the sides and ends of the forms were removed. The slabs were kept covered with wet burlap for 12 days. On the thirteenth day after casting the decking of the form was dropped and the test specimens removed from the steel stringers. The slab sections, which had been separated by the swelling of the wooden separators in the forms, were lifted from the I beams by a specially constructed steel cradle and a chain hoist suspended from a movable steel A frame. The sections were placed in the 8-foot spaces between the forms, resting on the narrow-gage rails, which served as sills and supported the slabs clear of the ground. The form was then reassembled and was ready to be used again the following day.

FLEXURE TESTS ON 3 BY 6 FOOT SLABS

Tests for flexural strength were made on four 3 by 6 foot sections of each large slab. All of these specimens were tested at the age of 22 days, the bottom of the slab being in tension. The machine used for the testing of these specimens is described below. On the twenty-second day after casting, the four sections not containing steel reinforcement (i. e., sections 2, 4, 5, and 7 in fig. 1) were removed from the storage tracks by means of the overhead A frame and hoist and were moved on the narrow-gage railway to the field testing machine.

This field testing machine is shown on the cover. It consisted of a structural-steel frame for supporting the specimen and for providing a reaction for the ball-bearing ratchet jack used to apply the load. Freedom of motion at the knife-edges was provided. The load was applied at the third points of the span and its magnitude was determined by a measurement of the deflection of a pair of steel beams previously calibrated.

In computing the modulus of rupture of the specimen, the dead loads due to the weight of certain parts of the testing machine and to the weight of the specimen itself were considered, as were the actual dimensions of the slab at the plane of rupture.

Comparative data from these flexure tests are shown in Table 3, and the average modulus of rupture for each test slab is shown in Figure 4. These data indicate that at the age of 22 days—

1. The concrete placed by the three vibrating methods had higher flexural strength than that placed by hand methods for all of the aggregates used.
2. The concrete placed by the Vibrolithic process had higher flexural strength than that placed by any of the other methods for all of the aggregates used.
3. In the case of the gravel concrete, that placed with the air hammer and that placed with the electric

TABLE 3.—*Values of modulus of rupture of 3 by 6 foot slabs tested at age of 22 days; span, 60 inches, load applied at third points*

GRAVEL AGGREGATE

Type	Slab No.	Section No.	Modulus of rupture	Average modulus of rupture	Average weight per cubic foot
			Lbs. per sq. in.	*Lbs. per sq. in.*	*Pounds*
Hand placed	12	2	468	465	144.8
		4	474		
		5	477		
		7	442		
	23	2	306	373	145.7
		4	418		
		5	376		
		7	392		
Vibrolithic process	4	2	488	535	147.4
		4	563		
		5	569		
		7	521		
	15	2	519	554	149.9
		4	576		
		5	570		
		7	553		
Electric tamper	7	2	478	435	147.9
		4	442		
		5	440		
		7	379		
	18	2	552	504	147.6
		4	521		
		5	499		
		7	443		
Air hammer	10	2	509	466	147.9
		4	459		
		5	437		
		7	457		
	21	2	490	476	147.5
		4	469		
		5	490		
		7	454		

LIMESTONE AGGREGATE

Type	Slab No.	Section No.	Modulus of rupture	Average modulus of rupture	Average weight per cubic foot
Hand placed	13	[1]2	441	452	148.1
		[1]4	457		
		[1]5	485		
		[1]7	427		
	24	[1]2	431	436	149.8
		[1]4	448		
		5	451		
		[1]7	413		
Vibrolithic process	5	2	614	579	151.8
		4	542		
		5	618		
		7	542		
	16	2	545	549	152.5
		4	525		
		5	581		
		7	546		
Electric tamper	8	2	589	574	151.2
		4	599		
		5	580		
		7	527		
	19	2	548	502	153.1
		4	515		
		5	489		
		7	457		
Air hammer	11	2	548	541	151.9
		4	586		
		5	517		
		7	512		
	22	2	538	472	150.4
		4	458		
		5	443		
		7	452		

HAYDITE AGGREGATE

Type	Slab No.	Section No.	Modulus of rupture	Average modulus of rupture	Average weight per cubic foot
Hand placed	3	[2]2	417	456	104.1
		4	483		
		5	482		
		[2]7	443		
	14	[2]2	441	450	104.1
		[2]4	441		
		[2]5	457		
		[2]7	459		
Vibrolithic process	6	2	463	507	105.5
		4	544		
		5	557		
		7	463		
	17	2	538	576	109.3
		4	550		
		5	557		
		7	659		
Electric tamper	9	2	425	426	105.4
		4	419		
		5	419		
		7	441		
	25	2	537	562	108.8
		4	574		
		5	592		
		7	547		

[1] Slight honeycomb in bottom. [2] Slight honeycomb.

tamper had approximately equal flexural strengths. In the case of the concrete in which crushed limestone was used as the coarse aggregate, that placed with the electric tamper had a flexural strength intermediate between that placed by the Vibrolithic process and that placed with the air hammer.

These indications are of course limited to the flexural tests at 22 days and do not take into consideration

TABLE 4.—*Values of modulus of rupture of 8 by 8 by 36 inch sawed beams; span, 30 inches, load applied at third points*

GRAVEL AGGREGATE

Type	Slab No.	Section No.	Modulus of rupture in pounds per square inch							
			28 days	Average for slab	90 days	Average for slab	180 days	Average for slab	365 days	Average for slab
Hand placed	12	2	------	-----	-----	701	627	610	553	580
		7	------		701		593		606	
	23	2	------	427	576	554	434	476	555	547
		7	427		532		519		539	
Vibrolithic process	4	2	------	609	849	732	620	636	676	716
		7	609		615		653		755	
	15	2	------	-----	-----	661	750	687	718	738
		7	------		661		625		759	
Electric tamper	7	2	------	-----	768	627	589	546	615	532
		7	------		486		503		449	
	18	2	------	-----	-----	547	550	548	623	628
		7	------		547		546		633	
Air hammer	10	2	------	475	-----	627	623	545	774	590
		7	475		627		467		407	
	21	2	------	511	742	644	642	614	776	736
		7	511		547		586		696	

LIMESTONE AGGREGATE

Type	Slab No.	Section No.	28 days	Average for slab	90 days	Average for slab	180 days	Average for slab	365 days	Average for slab
Hand placed	13	2	------	-----	681	669	521	423	480	523
		4	------		620		360		700	
		5	------		606		360		423	
		7	------		764		452		490	
	24	2	518	461	529	606	480	506	666	691
		4	------		639		515		652	
		5	------		644		503		626	
		7	404		610		525		820	
Vibrolithic process	5	2	673	616	797	773	773	710	696	710
		4	------		801		736		737	
		5	------		791		768		714	
		7	560		701		564		691	
	16	2	623	560	596	778	561	660	536	650
		4	------		885		657		643	
		5	------		876		735		739	
		7	498		754		688		684	
Electric tamper	8	2	704	630	783	752	692	647	788	745
		4	------		818		735		830	
		5	------		734		638		686	
		7	555		672		523		676	
	19	2	517	518	716	669	736	645	761	684
		4	------		661		617		710	
		5	------		700		652		596	
		7	518		598		574		667	
Air hammer	11	2	596	495	833	731	763	680	806	720
		4	------		666		652		787	
		5	------		752		634		635	
		7	393		673		672		650	
	22	2	525	501	625	622	731	619	730	678
		4	------		700		499		827	
		5	------		623		617		580	
		7	477		538		628		572	

HAYDITE AGGREGATE

Type	Slab No.	Section No.	28 days	Average for slab	90 days	Average for slab	180 days	Average for slab	365 days	Average for slab
Hand placed	3	2	396	401	519	469	571	496	284	305
		4	432		517		481		365	
		5	358		406		506		224	
		7	419		435		424		348	
	14	2	531	496	683	718	366	372	243	264
		4	487		758		385		259	
		5	468		788		358		280	
		7	500		643		380		274	
Vibrolithic process	6	2	526	457	580	604	491	476	245	232
		4	461		522		503		252	
		5	441		638		505		244	
		7	400		687		406		186	
	17	2	646	653	550	557	570	555	285	275
		4	628		562		553		270	
		5	678		542		-----		227	
		7	659		575		541		318	
Electric tamper	9	2	643	573	546	541	411	433	245	250
		4	549		531		485		207	
		5	576		542		464		301	
		7	524		545		372		248	
	25	2	499	503	631	553	378	407	394	355
		4	488		520		375		295	
		5	514		515		394		370	
		7	511		546		482		362	

possible effects of variations in temperature and humidity during this period.

FLEXURE TESTS ON 8 BY 8 BY 36 INCH SAWED BEAMS

The specimens used in these tests were obtained by sawing into beams the fragments of the 3 by 6 foot slabs used in the 22-day flexure tests. This operation was performed with a high-speed inserted-tooth carborundum saw which in general was very satisfactory, except that the cutting in the siliceous aggregates was somewhat slow.

applied loads and was based on the actual ruptured section of the specimen.

The flexural strengths of the individual specimens are given in Table 4, as well as averages for the beams from each section of each slab. Figure 6 shows graphically the effects of age, method of placing, and of aggregates on the flexural strength of these specimens.

In order to study the effect of the various methods of placing on the flexural strength of concrete of each material, a summary of the data in Table 4 was pre-

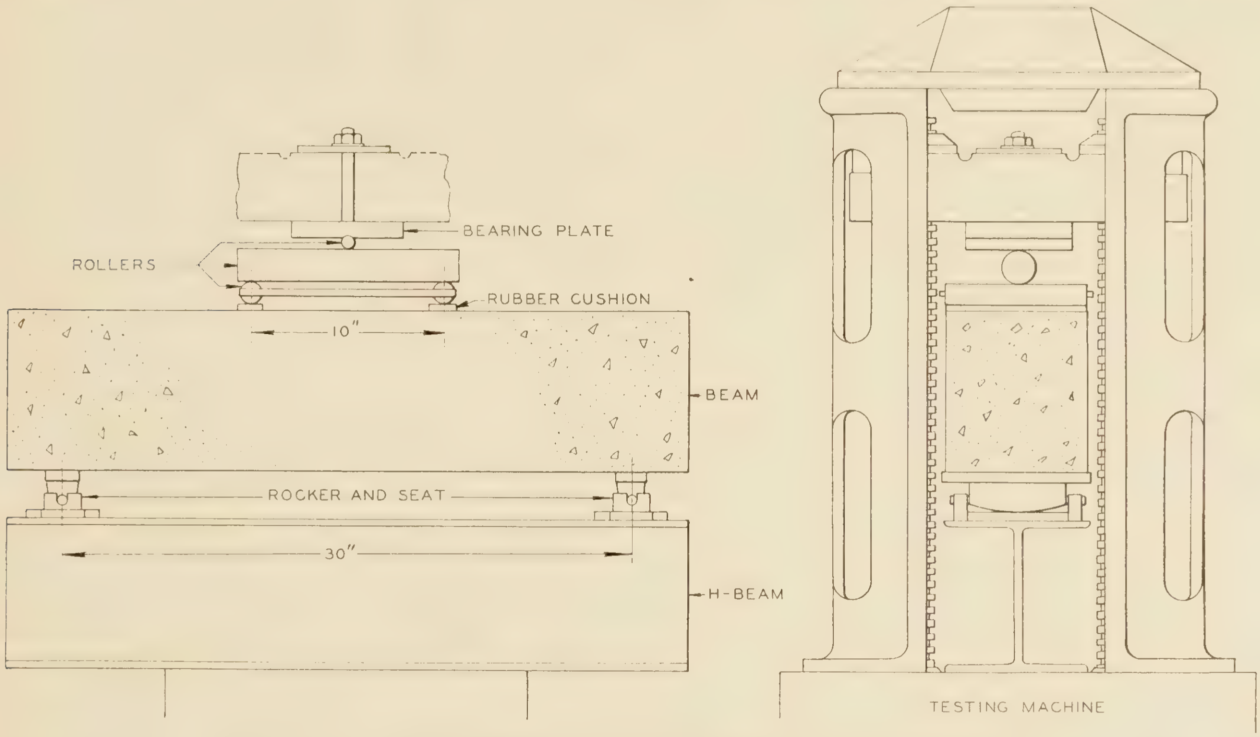


FIGURE 5.—APPARATUS USED IN MAKING FLEXURE TESTS ON SAWED BEAMS

The program called for the testing of four beams from each large slab at each of four test periods (28 and 90 days, 6 months, and 1 year). Because of the difficulty of sawing the specimens of gravel concrete and other unavoidable delays, it was not possible to prepare and test this number of specimens, and it will be noted in the tabulated data that the number of specimens varies, particularly in the 28-day tests.

Another difficulty which was not foreseen was a considerable variation in the moisture condition of the sawed specimens tested at the early ages. The sawing process requires a free flow of water over the specimen, and at the early ages some of the specimens were taken almost directly from the saw to the testing machine. Moreover, the specimens for the tests at 28 days, 90 days, and 6 months were stored in the open air without protection from the time of sawing until the time of test. In the case of the 1-year test specimens, the sawing was completed well in advance of the date of test and the specimens were stored in a metal building for at least 10 weeks before testing.

The apparatus used for this test is shown in Figure 5. The load was applied at the third points of the span, which was 30 inches. Tension was produced in the bottom of the specimen as cast. The conventional testing machine in which the apparatus was set up was operated slowly with a handwheel. The modulus of rupture was calculated from the combined dead and

pared, considering the strength of the hand-placed concrete as 100 per cent in each case. The results of this summary are shown in Table 5.

The indications of these figures are in rather striking agreement with those of the tests of the 3 by 6 foot slabs at 22 days. It will be noted that the vibrated concrete generally shows higher flexural strength than does the hand-placed concrete. This is most noticeable in the case of the specimens having the limestone aggregate, and is also clearly indicated by the specimens having gravel aggregate. In the case of the specimens in which Haydite was used, the majority of tests indicate a slight superiority on the part of the hand-placed concrete. It is also shown that of the three vibrating methods used, the Vibrolithic process yielded concrete of the greatest flexural strength.

Comment has been made on the possible effect of the moisture condition of the specimens at the time of test on their flexural strength. It is believed that the apparent general retrogression in strength between 90 and 180 days is due to the fact that the specimens were in a drier condition at 180 than at 90 days. Special care was taken with the 1-year specimens, as has been noted.

In general, the data show a normal increase in flexural strength except in the case of the Haydite concrete specimens, where a sharp drop in flexural strength took place between the 6-month and 1-year tests. The cause of this has not been determined.

FIGURE 6.—EFFECTS OF AGE, METHOD OF PLACING, AND TYPE OF AGGREGATE ON FLEXURAL STRENGTH OF CONCRETE. VALUES SHOWN ARE MEAN VALUES OF MODULUS OF RUPTURE FOR SAWED BEAMS FROM TWO SLABS OF EACH TYPE AS GIVEN IN TABLE 4. DASH LINES INDICATE MEAN VALUES FOR LARGE SLABS BROKEN AT 22 DAYS

TABLE 5.—*Flexural strength of 8 by 8 by 36 inch sawed beams, expressed as a percentage of the strength of the beams taken from hand-placed concrete*

GRAVEL AGGREGATE

Age	Hand placed	Vibro-lithic process	Electric tamper	Air hammer
DAYS	*Per cent*	*Per cent*	*Per cent*	*Per cent*
28	100	143		115
90	100	111	94	101
180	100	122	100	107
365	100	129	103	118
Average	100	126	99	110

LIMESTONE AGGREGATE

Age	Hand placed	Vibro-lithic process	Electric tamper	Air hammer
28	100	128	125	108
90	100	122	111	106
180	100	147	139	140
365	100	112	118	115
Average	100	128	123	117

HAYDITE AGGREGATE

Age	Hand placed	Vibro-lithic process	Electric tamper	Air hammer
28	100	124	120	
90	100	98	92	
180	100	119	97	
365	100	89	106	
Average	100	107	104	

After the 1-year tests, additional specimens were sawed from the slabs of Haydite concrete, dried for a period of 25 days in a boiler room and tested at the age of 15 months. At this time the weight appeared to be constant. The 15-month tests showed a decrease in flexural strength over that of the specimens tested at one year. This decrease averaged about 20 per cent of the 1-year strength. Unfortunately there was not sufficient concrete available to attempt to determine the cause of this peculiar retrogression in flexural strength, but it is suspected that it may be related to the moisture-retaining properties of this material, and further investigation seems advisable.

COMPRESSIVE STRENGTH OF 6 BY 8 INCH DRILLED CORES

Tests for compressive strength were made on 6 by 8 inch drilled cores at ages of 28 days, 90 days, and 1 year. These cores were drilled with a conventional steel shot core drill from fragments of the 3 by 6 foot slabs used for the 22-day flexure tests. One core was drilled from each of the four sections of each 6 by 22 foot slab for each of the three ages at which compression tests were made. Within 24 hours after drilling the cores were capped with neat cement and stored in the open air, adjacent to the slabs from which they were taken, until they were tested. Correction was made for the length of the core in calculating the unit compressive strength.

The data from these tests are given in Table 6 and the average values are shown graphically in Figure 7.

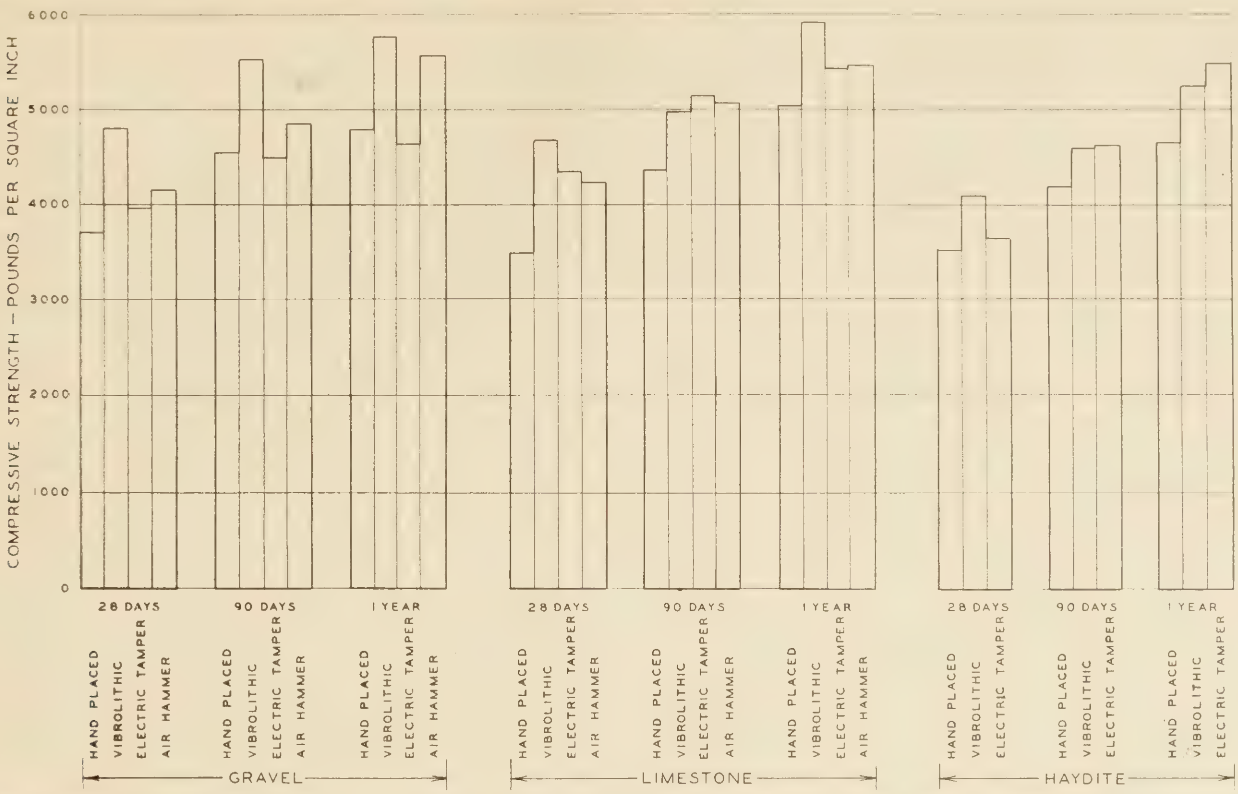


FIGURE 7.—EFFECTS OF AGE, METHOD OF PLACING, AND TYPE OF AGGREGATE ON COMPRESSIVE STRENGTH OF DRILLED CORES

TABLE 6.—*Compressive strength of 6 by 8 inch drilled cores*

Slab No.	Aggregate	Method of placing	Compressive strength in pounds per square inch: 28 days	90 days	365 days
12, 23	Gravel	Hand placed	4,137; 3,310	4,830; 4,302	4,885; [1] 4,740
	Average		3,724	4,566	4,812
4, 15	Gravel	Vibrolithic process	4,760; 4,893	5,366; 5,675	5,915; 5,640
	Average		4,826	5,520	5,778
7, 18	Gravel	Electric tamper	3,975; 3,961	4,410; 4,583	4,525; 4,810
	Average		3,968	4,496	4,668
10, 21	Gravel	Air hammer	4,137; 4,173	4,920; 4,780	5,600; 5,580
	Average		4,155	4,850	5,590
13, 24	Limestone	Hand-placed	3,471; 3,532	4,614; 4,120	5,150; 4,900
	Average		3,502	4,367	5,025
5, 16	Limestone	Vibrolithic process	4,701; 4,652	4,684; [1] 5,278	5,995; 5,825
	Average		4,676	4,981	5,910
8, 19	Limestone	Electric tamper	4,444; 4,210	5,429; 4,844	5,915; [1] 4,955
	Average		4,327	5,136	5,435
11, 22	Limestone	Air hammer	4,527; 3,894	5,277; 4,810	5,440; [1] 5,485
	Average		4,210	5,044	5,462
3, 14	Haydite	Hand-placed	3,863; 3,136	4,375; 4,015	4,880; 4,485
	Average		3,500	4,195	4,682

[1] Average of 3 tests. All other values given are averages of 4 tests.

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TABLE 6.—*Compressive strength of 6 by 8 inch drilled cores*—Con.

Slab No.	Aggregate	Method of placing	Compressive strength in pounds per square inch: 28 days	90 days	365 days
6, 17	Haydite	Vibrolithic process	4,014; 4,164	4,590; 4,684	5,365; 5,110
	Average		4,090	4,637	5,238
9, 25	Haydite	Electric tamper	3,555; [1] 4,091	4,841; 4,428	5,558; 5,410
	Average		3,623	4,640	5,484

It will be noted from these data that the concrete, for all methods of placing and for all three aggregates, shows a progressive increase in strength with age. It is also evident that the mechanical methods of placing, in general, result in higher compressive strength than is obtained by hand placing and also that of the three mechanical methods used the Vibrolithic process gave concrete of the highest compressive strength.

In an effort to rate the four methods of placing used in this investigation, an average flexural strength and an average compressive strength was obtained for each method. In these average values are included all ages of test and all aggregates. The average strength is expressed, in each case, as a percentage of the strength of the hand-placed concrete of comparable age and aggregate. The result of this comparison is as follows:

	Hand placed	Vibrolithic process	Electric tamper	Air hammer
Flexural strength[1] (sawed beams)	100	120	108	113
Compressive strength[1] (drilled cores)	100	119	109	113

[1] Strength expressed as a percentage.

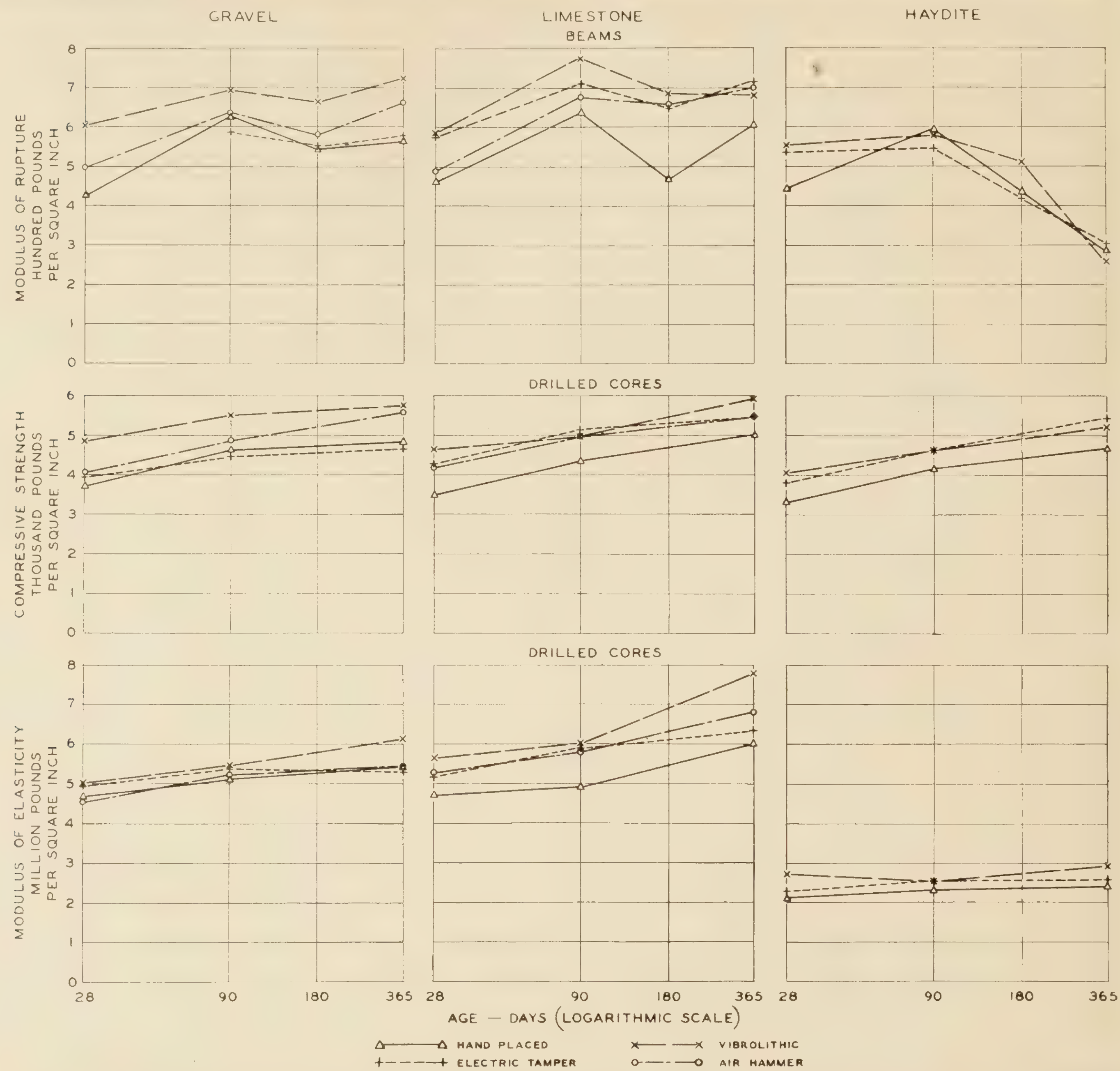


Figure 8.—Résumé of Strength Data, Showing Variation of Flexural Strength of Sawed Beams, Compressive Strength of Drilled Cores, and Modulus of Elasticity of Drilled Cores, with Increasing Age of Specimen

These values show rather strikingly that the method of placing influences the flexural and the compressive strength to an equal degree. All of the data are in general accord as to the effect of the methods of placing on the strength of the concrete.

The concrete containing the Haydite aggregate compares very favorably in compressive strength with that containing the other aggregates and no retrogression in compressive strength was found to occur. A limited number of cores were tested at the age of 15 months. One half of these were in a saturated condition and the other half were thoroughly dried. No marked difference in strength between the wet and the dry specimens was found. Moreover, no appreciable change in compressive strength between the 1-year and 15-month periods was observed.

In comparing aggregates it should be remembered that the cement factor of the concrete containing the Haydite was about one and one-half times that of the concrete containing the other aggregates. If the comparison is made on the basis of equal cement factors (see Table 2) and all ages and methods of placing are averaged, it will be found that, from the standpoint of compressive strength only, the following relation exists:

Type of aggregate:	Compressive strength in pounds per square inch per bag of cement in a cubic yard of concrete
Gravel	929
Limestone	923
Haydite	560

If the reciprocals of the values of compressive strength tabulated above are obtained and multiplied by 1,000, an index is obtained which gives the number of sacks per cubic yard required per 1,000 pounds unit compressive strength in the concrete. Values of the index thus obtained are as follows:

Type of aggregate:	Bags per cubic yard for each 1,000 pounds per square inch compressive strength
Gravel	1.076
Limestone	1.083
Haydite	1.786

These values are, of course, useful only for comparing the costs of concrete of a given compressive strength when made of the materials used in this investigation.

It should also be remembered that the weight of the concrete containing the Haydite aggregate is about 105 to 110 pounds per cubic foot, as compared with a weight per cubic foot of from 145 to 150 pounds for the concrete containing the other aggregates (see Table 3), and it will probably be found in the majority of cases that decreases in slab weight are of more economic significance than are increases in the cost of construction of the slab itself.

MODULUS OF ELASTICITY

At each of the periods at which compressive strength tests were made the modulus of elasticity was determined for each of the cores. The loads were applied in a universal testing machine operated by a handwheel. The deformations were measured with a Martens type of extensometer operating on a 4-inch gage length.

TABLE 7.—*Average values of modulus of elasticity of 6 by 8 inch drilled cores*

Slab No.	Aggregate	Method of placing	Initial modulus of elasticity: 28 days	Initial modulus of elasticity: 90 days	Initial modulus of elasticity: 365 days
			Lbs. per sq. in.	*Lbs. per sq. in.*	*Lbs. per sq. in.*
12	Gravel	Hand placed	4,925,000	6,105,000	5,625,000
23	Gravel	Hand placed	4,377,000	4,165,000	[1] 5,147,000
	Average		4,651,000	5,135,000	5,386,000
4	Gravel	Vibrolithic process	5,110,000	5,876,000	6,619,000
15	Gravel	Vibrolithic process	4,937,000	5,042,000	5,677,500
	Average		5,023,500	5,459,000	6,148,250
7	Gravel	Electric tamper	4,280,000	5,742,000	5,549,750
18	Gravel	Electric tamper	5,680,000	5,102,000	5,100,000
	Average		4,980,000	5,422,000	5,324,875
10	Gravel	Air hammer	4,580,000	5,110,000	4,759,250
21	Gravel	Air hammer	4,491,000	5,225,000	6,030,000
	Average		4,535,500	5,167,500	5,394,625
13	Limestone	Hand placed	4,900,000	5,135,000	6,410,000
24	Limestone	Hand placed	4,490,000	4,617,000	5,620,000
	Average		4,695,000	4,876,000	6,015,000
5	Limestone	Vibrolithic process	5,085,000	6,007,000	7,889,750
16	Limestone	Vibrolithic process	6,205,000	5,970,000	7,672,500
	Average		5,645,000	5,988,500	7,781,125
8	Limestone	Electric tamper	5,305,000	6,380,000	6,750,000
19	Limestone	Electric tamper	5,010,000	5,350,000	[1] 5,893,000
	Average		5,157,500	5,865,000	6,321,500
11	Limestone	Air hammer	5,700,000	6,190,000	7,130,000
22	Limestone	Air hammer	4,815,000	5,415,000	[1] 6,426,000
	Average		5,257,500	5,802,500	6,778,000
3	Haydite	Hand placed	2,240,000	2,401,000	2,450,500
14	Haydite	Hand placed	2,025,000	2,290,000	2,437,000
	Average		2,132,500	2,345,500	2,443,750
6	Haydite	Vibrolithic process	2,655,000	2,642,000	3,038,000
17	Haydite	Vibrolithic process	2,855,000	2,394,000	2,935,000
	Average		2,755,000	2,518,000	2,986,500
9	Haydite	Electric tamper	2,230,000	2,385,000	2,600,000
25	Haydite	Electric tamper	2,352,000	2,757,000	2,660,000
	Average		2,291,000	2,571,000	2,630,000

[1] Average of 3 tests. All other values given are averages of 4 tests.

(For a description of an instrument of this type see PUBLIC ROADS, vol. 9, No. 8, October, 1928, The Modulus of Elasticity of Cores from Concrete Roads. by A. N. Johnson.) The precision of the deformation measurements is very high with this apparatus, and for the concrete used in these tests the stress-strain relation was linear up to unit stresses of several hundreds of pounds per square inch. It is the slope of this initial curve that is referred to in this report as the modulus of elasticity.

The average values for each group of four cores obtained for each age of test from each of the large 6 by 22 foot slabs are shown in Table 7, and these averages are shown graphically in Figure 8, where the effect of age on the modulus of elasticity is indicated.

This graph also contains the data from the flexure tests on the sawed beams and from the compression tests on drilled cores plotted on a logarithmic time scale and gives a good summary picture of the data which have been thus far discussed.

It will be noted in this figure that generally the value of modulus of elasticity increases with age. This effect is most marked in the case of the concrete containing the limestone aggregate and is barely noticeable in that containing the Haydite.

The modulus of elasticity of the concrete containing the Haydite aggregate is generally of about one-half the magnitude of that for the concrete containing the other aggregates. This was consistently true throughout the considerable number of tests made (72 Haydite cores). Although the slope of the load-deformation curve was radically different for this concrete, the relation appeared to be linear for moderate unit stresses except in the case of the 28-day tests, where curvature began at very low loads.[1]

The individual curves have been omitted because of their large number.

COMPRESSIVE STRENGTH OF 6 BY 12 INCH CONTROL CYLINDERS

At the time that this investigation was planned it was realized that, during the construction and curing of the several slabs, there would be weather variations and that these might have an effect on the strength of the concrete. Any appreciable effect on strength would have to be considered in drawing conclusions about the variables under study. It was believed that if a group of 6 by 12 inch cylinders were made out of the concrete that went into each slab and that if these were exposed to the same conditions of curing and weather as the slabs themselves, then for each type of aggregate, variations in strength between groups of cylinders would reflect the influence of the variations in temperature and humidity to which the concrete had been exposed.

To provide these data, five 6 by 12 inch cylinders were made from concrete from sections 2, 4, 5, 6, and 8, respectively, of each large slab. These specimens were fabricated in accordance with standard practice in steel molds resting on and covered with glass plates. When 24 hours old they were removed from the molds, capped, and placed under the wet burlap which covered the slab from which they were made. The cylinders remained on the slabs until the burlap was removed and were then stored in the vicinity until they were ready for test at the age of 28 days.

All of the data pertaining to the cylinders are given in Table 8. The water-cement ratio shown is a calcu-

[1] For a discussion of the effect of this low modulus of elasticity, see Jour. Amer. Conc. Inst., vol. 2, No. 2, October, 1930, p. 151, Construction and Design Features of Haydite Concrete, by F. E. Richart and V. P. Jensen.

lated value and the air temperature is a mean value for the 10-day period immediately following the placing of the concrete. Mean compressive strengths are shown for each group of five specimens, and the individual variations from the group means are tabulated.

TABLE 8.—*Test data pertaining to the 6 by 12 inch control cylinders*

GRAVEL AGGREGATE

Slab No.	Section No.	Water-cement ratio	10-day temperature	Compressive strength: Individual	Compressive strength: Average	Variation from group average
			° F.	Lbs. per sq. in.	Lbs. per sq. in.	Per cent
	2			3,450		8.6
	4			3,860		2.2
4	5	0.86	69.2	3,900	3,776	3.3
	6			3,640		3.6
	8			4,030		6.7
	2			3,470		5.0
	4			3,110		5.9
7	5	.86	53.2	3.350	3.306	1.3
	6			3,240		2.0
	8			3,360		1.6
	2			4,135		7.4
	4			3.905		1.4
10	5	.85	58.3	3,895	3,850	1.1
	6			3.760		2.3
	8			3,555		7.7
	2			3,706		.2
	4			3,788		2.4
12	5	.86	62.7	3,843	3,698	3 9
	6			3,745		1.1
	8			3,406		7.9
	2			3,851		2.5
	4			4.256		13.3
15	5	.86	64.7	3,662	3,757	2.3
	6			3,378		10.1
	8			3,638		3.2
	2			3,246		.4
	4			3,229		.2
18	5	.86	60.9	3,248	3,234	.4
	6			3,246		.4
	8			3,199		1.1
	2			3,791		5.2
	4			3,451		4.3
21	5	.86	43.5	3,617	3,605	.3
	6			3,681		2.1
	8			3,483		3.4
	2			2,964		2.5
	4			2,942		3.2
23	5	.85	51.6	3,258	3,040	7.2
	6			2,990		1.6
	8			3,045		.2

LIMESTONE AGGREGATE

Slab No.	Section No.	Water-cement ratio	10-day temperature	Compressive strength: Individual	Compressive strength: Average	Variation from group average
	2			3,200		3.0
	4			3,300		0
5	5	0.90	64.8	2,940	3,300	10.9
	6			3,660		10.9
	8			3,400		3.0
	2			3,645		.7
	4			3,610		1.7
8	5	.91	52.8	4,025	3,672	9.6
	6			3,490		5.0
	8			3,590		2.2
	2			3,345		6.3
	4			3,700		3.6
11	5	.91	60.9	3,675	3,570	2.9
	6			3,580		.3
	8			3,550		.6
	2			4,380		13.3
	4			3,440		11.0
13	5	.91	62.9	4,065	3,865	5.2
	6			4,030		4.3
	8			3,410		11.8
	2			3,499		11.0
	4			3,175		.7
16	5	.91	64.1	3,056	3,153	3.1
	6			2,263		28.2
	8			3,773		19.7
	2			3,316		7.3
	4			2,742		11.3
19	5	.91	54.8	3,087	3,090	.1
	6			3,412		10.4
	8			2,893		6.4
	2			3,023		2.1
	4			3,190		7.7
22	5	.91	47.4	2,891	2,961	2.4
	6			2,783		6.0
	8			2,920		1.4
	2			3.520		15.4
	4			2,980		2.0
24	5	.91	47.4	3,117	3,051	2.2
	6			2,888		5.3
	8			2,752		9.8

TABLE 8.—*Test data pertaining to the 6 by 12 inch control cylinders*—Continued

HAYDITE AGGREGATE

Slab No.	Section No.	Water-cement ratio	10-day temperature	Compressive strength: Individual	Compressive strength: Average	Variation from group average
			° F.	Lbs. per sq. in.	Lbs. per sq. in.	Per cent
	2			4,050		7.9
	4			4,320		1.8
3	5	0.88	71.8	4,380	4,398	.4
	6			4,280		2.7
	8			4,960		12.8
	2			4,750		2.3
	4			5,030		3.4
6	5	.76	56.6	4 860	4,864	.1
	6			4,860		.1
	8			4,820		.9
	2			4,730		2.4
	4			5,120		5.6
9	5	.79	55.0	4,385	4,847	9.5
	6			5,390		11.2
	8			4,610		4.9
	2			4,122		.6
	4			4,361		6.4
14	5	.78	64.1	4,343	4,098	6.0
	6			4,407		7.5
	8			3,259		20.5
	2			4,404		5.6
17	4	.78	65.6	5,172	4,665	10.9
	5			4,217		9.6
	6			4,867		4.3
	2			4,673		7.2
	4			5,617		11.5
25	5	.73	38.0	5,090	5,037	1.1
	6			4,693		6.8
	8			5,112		1.5

A study of these data was made in order to draw some conclusion as to the reliability of the indicated strengths for use as a basis upon which to compare the strength values obtained from the other tests.

It will be noted that there is a general relation between the compressive strengths and the water-cement ratio. If all of the values for a particular aggregate are averaged, it will be found that the following relation exists:

Aggregate	Average water-cement ratio	Average compressive strength in pounds per square inch	Average cement factor in bags per cubic yard
Crushed stone	0.91	3,333	5.25
Gravel	.86	3,534	5.11
Haydite	.79	4,652	8.00

The variation in strength of the individual cylinders from the respective group averages was computed, and these data are included in Table 8, as previously noted. From these variations it was found that—

96 per cent of the values show variations of 15 per cent or less;

84 per cent of the values show variations of 10 per cent or less;

75 per cent of the values show variations of 7.5 per cent or less;

59 per cent of the values show variations of 5 per cent or less.

The mean variation for all of the specimens from their respective groups averages is 5.2 per cent.

This analysis indicates that there is no abnormal variation in the strength data for the control cylinders when the number of specimens is considered. It also emphasizes the desirability of having a greater number of specimens than were available in these tests, for an investigation where the close determination of strength is of importance. In spite of careful fabrication and attention to the details of testing technique, unaccountable strength variations will appear in the data. Occasionally these are of rather large magnitude, but unless there are definite grounds in the history of the specimen for rejecting the data they must be retained and included in the averages. Their effect may be minimized by conducting a large number of tests.

In order to examine the data from the cylinders with respect to effect of temperature, as indicated by the 10-day mean air temperature, the average compressive strengths were arranged in the order of descending temperature values in Table 9. While there appears to be some indication of an increase in compressive strength with an increase in the average air temperature over the 10-day period in the case of the specimens containing crushed stone and gravel aggregates, the relation is not established and there is no such indication in the case of the specimens containing Haydite. The conclusion is drawn, therefore, that the data from the control specimens are normal in character although

TABLE 9.—*Average compressive strength of groups of control cylinders compared with corresponding 10-day mean air temperature*

Gravel aggregate		Crushed-stone aggregate		Haydite aggregate	
Temperature	Strength	Temperature	Strength	Temperature	Strength
° F.	*Lbs. per sq. in.*	° F.	*Lbs. per sq. in.*	° F.	*Lbs. per sq. in.*
69.2	3,776	64.8	3,300	71.8	4,398
64.7	3,757	64.1	3,153	65.6	4,665
62.7	3,698	62.9	3,865	64.1	4,098
60.9	3,234	60.9	3,570	56.6	4,864
58.3	3,850	54.8	3,090	55.0	4,847
53.2	3,306	52.8	3,672	38.0	5,037
51.6	3,040	47.4	3,051		
43.5	3,605	47.4	2,961		

TABLE 10.—*Compressive strengths of drilled cores and flexural strengths of sawed beams, expressed as percentages of strengths of corresponding control cylinders*

GRAVEL

Slab No.	Type	A=average compressive strength of control cylinders	Drilled cores; average compressive strength, in pounds per square inch: 28 days	Percentage of A	90 days	Percentage of A	1 year	Percentage of A	Sawed beams; average modulus of rupture, in pounds per square inch: 28 days	Percentage of A	90 days	Percentage of A	180 days	Percentage of A	1 year	Percentage of A
		Lbs. per sq. in.														
12	Hand placed	3,698	4,137	111.9	4,830	130.6	4,885	132.1			701	19.0	610	16.5	580	15.7
23		3,040	3,310	108.9	4,302	141.5	4,775	157.1	427	14.0	554	18.2	476	15.6	547	18.0
	Average			110.4		136.0		144.6		14.0		18.6		16.0		16.8
4	Vibrolithic	3,776	4,760	126.0	5,366	142.1	5,915	156.6	609	16.1	732	19.4	636	16.8	716	19.0
15		3,757	4,893	130.2	5,675	151.0	5,640	150.1			661	17.6	687	18.3	738	19.6
	Average			128.1		146.5		153.4		16.1		18.5		17.6		19.3
7	Electric tamper	3,306	3,975	120.2	4,410	133.4	4,525	136.9			627	19.0	546	16.5	532	16.1
18		3,234	3,961	122.5	4,583	141.7	4,810	148.7			547	16.9	548	16.9	628	19.4
	Average			121.4		137.5		142.8				18.0		16.7		17.8
10	Air hammer	3,850	4,137	107.4	4,920	127.8	5,600	145.4	475	12.3	627	16.3	545	14.2	590	15.3
21		3,605	4,173	115.8	4,780	132.6	5,580	154.8	511	14.2	644	17.9	614	17.0	736	20.4
	Average			111.6		130.2		150.1		13.2		17.1		15.6		17.8

LIMESTONE

Slab No.	Type	A	28 days	Percentage of A	90 days	Percentage of A	1 year	Percentage of A	28 days	Percentage of A	90 days	Percentage of A	180 days	Percentage of A	1 year	Percentage of A
13	Hand placed	3,865	3,471	89.8	4,614	119.4	5,150	133.2			669	17.3	423	10.9	523	13.5
24		3,051	3,532	115.8	4,120	135.0	4,900	160.6	461	15.1	606	19.9	506	16.6	691	22.6
	Average			102.8		127.2		146.9		15.1		18.6		13.8		18.0
5	Vibrolithic	3,300	4,701	142.4	4,684	141.9	5,995	181.7	616	18.7	773	23.4	710	21.5	710	21.5
16		3,153	4,652	147.5	5,278	167.4	5,825	184.7	560	17.8	778	24.7	660	20.9	650	20.6
	Average			144.9		154.6		183.2		18.2		24.0		21.2		21.0
8	Electric tamper	3,672	4,444	121.0	5,429	147.8	5,915	160.9	630	17.1	752	20.5	647	17.6	745	20.3
19		3,090	4,210	136.2	4,844	156.8	4,955	160.3	518	16.8	669	21.6	645	20.9	684	22.1
	Average			128.6		152.3		160.6		17.0		21.0		19.2		21.2
11	Air hammer	3,570	4,525	126.8	5,277	147.8	5,440	152.4	495	13.9	731	20.5	680	19.0	720	20.2
22		2,961	3,894	131.5	4,810	162.4	5,485	185.2	501	16.9	622	21.0	619	20.9	678	22.9
	Average			129.2		155.1		168.8		15.4		20.7		19.9		21.6

HAYDITE

Slab No.	Type	A	28 days	Percentage of A	90 days	Percentage of A	1 year	Percentage of A	28 days	Percentage of A	90 days	Percentage of A	180 days	Percentage of A	1 year	Percentage of A
3	Hand placed	4,398	3,863	87.8	4,375	99.5	4,880	110.9	401	9.1	469	10.7	496	11.3	305	6.9
14		4,098	3,136	76.5	4,015	98.0	4,485	109.4	496	12.1	718	17.5	372	9.1	264	5.3
	Average			82.2		98.8		110.2		10.6		14.1		10.2		6.1
6	Vibrolithic	4,864	4,014	82.5	4,590	94.4	5,365	110.3	457	9.4	604	12.4	476	9.8	232	4.8
17		4,665	4,165	89.3	4,684	100.4	5,110	109.5	653	14.0	557	11.9	555	11.9	275	5.9
	Average			85.9		97.4		109.9		11.7		12.2		10.8		5.4
9	Electric tamper	4,847	3,555	73.3	4,841	99.9	5,560	114.7	573	11.8	541	11.2	433	8.9	250	5.2
25		5,037	4,091	81.2	4,428	87.9	5,410	107.4	503	10.0	553	11.0	407	8.1	355	7.0
	Average			77.2		93.9		111.0		10.9		11.1		8.5		6.1

somewhat deficient in number, and that the lack of an adequate number of specimens probably accounts for the failure to establish the relation between curing period temperature and compressive strength.

In Table 10 and Figure 9 are shown the compressive and flexural strength data obtained from the various slabs at ages of 1, 3, 6, or 12 months. These data are expressed as percentages of the average compressive strength of the group of five cylinders tested for each of the particular slabs involved. In other words, the strength of this group at 28 days is taken as 100 per cent and all other test values related to it. It should be observed that Table 10 and Figure 9 do not afford a direct comparison of the strength of concretes made with different aggregates. They serve rather to indicate the extent to which the strength characteristics found in the control cylinders were realized in the finished concrete.

It will be noted that when the relative strengths of the concretes containing the three different aggregates are taken into account the trends and relationships shown by Figure 9 are practically the same as those expressed by the strength data before they were modified by the data regarding the control specimens given in Figure 8.

RESULTS OF STRENGTH TESTS SUMMARIZED

The data obtained from tests of flexural and compressive strength indicate that—

1. Mechanical methods of placing resulted in greater flexural and compressive strengths than hand placing in the case of the concretes containing gravel and crushed-stone aggregates.

2. In the case of the concrete containing the Haydite aggregate, mechanical methods of placing produced concrete of higher compressive strength than hand

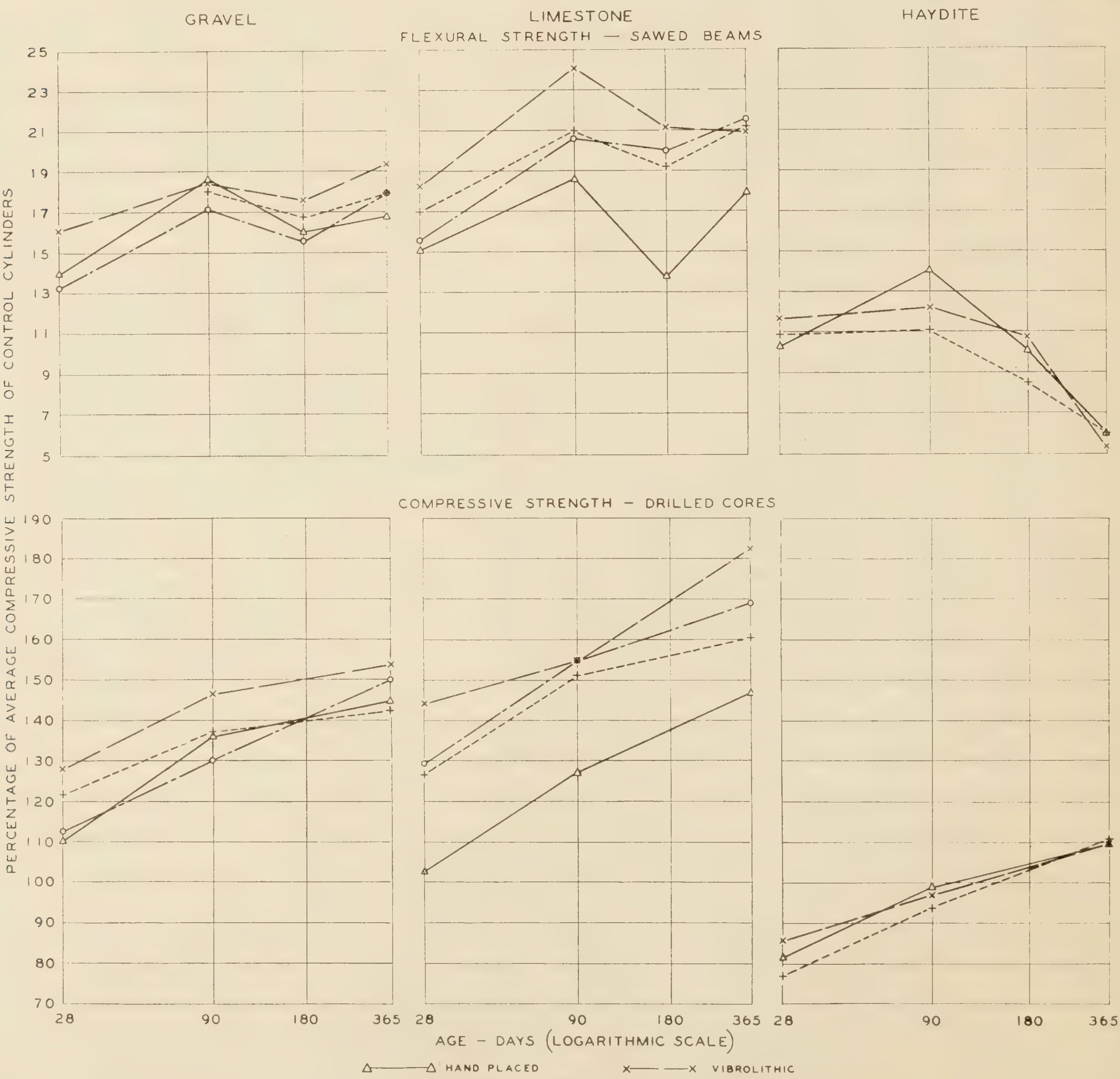


Figure 9.—Flexural Strengths of Sawed Beams and Compressive Strengths of Drilled Cores, Expressed as Percentages of Compressive Strengths of Corresponding Control Cylinders

placing. The effect of vibration on the flexural strength seems to be uncertain, and this may be due to the tendency of the materials to segregate according to their specific gravities when subjected to vibration.

3. The increases in flexural and compressive strengths due to mechanical placing are considerably more marked in the case of the angular aggregate than they are in the case of the rounded aggregate.

4. There is a normal increase in compressive strength with age for all materials and methods of placing.

5. A general increase in flexural strength with age occurred with both the crushed-stone and gravel aggregates, while for some reason not disclosed by these tests a continued retrogression in flexural strength occurred after 90 days in the concrete containing Haydite aggregate.

6. Of the three methods of mechanical placing employed in this investigation, the Vibrolithic process had the greatest effect on strength.

BOND STRENGTH OF EMBEDDED STEEL BARS

In order to develop information concerning the effect of the aggregates and methods of placing on the bond strength of embedded steel, a rather comprehensive series of bond tests was included in the program. Reinforcing bars of two sizes, each size of both plain and deformed types, were embedded both vertically and horizontally in each of the 11 types of slab. In all, 176 tests were made. The construction of the specimens used in these tests was unique in some respects and for this reason will be described in some detail.

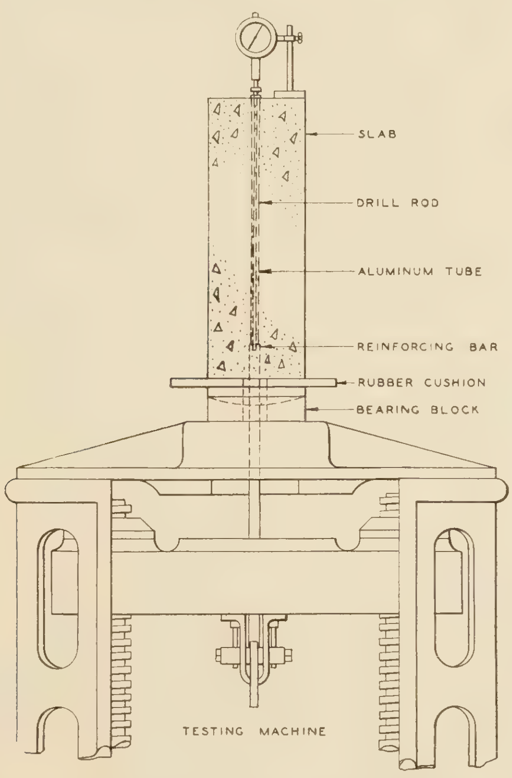


Figure 10.—Apparatus Used in Bond Tests of Bars Embedded Horizontally

Two-foot lengths of either deformed or plain round bars were embedded in sections 1 and 8 of each of the large slabs. The deformed bars were used in the first series of 11 slabs and the plain bars in the second series, with the exception that deformed bars were used in slabs 23 and 24, and plain bars in slabs 12 and 13. The 1-inch bars were placed in section 1 and the ½-inch bars in section 8 in all cases. The depth of embedment was 6½ inches for both the vertical and horizontal positions.

The vertical bars were passed through holes in the bottom of the slab form and were secured in such a position that the desired embedment was obtained. The upper end was capped with a large cork. After the finishing was completed and the concrete had hardened, the cork was removed and the end of the bar exposed. This permitted a measurement of slip to be made on these specimens.

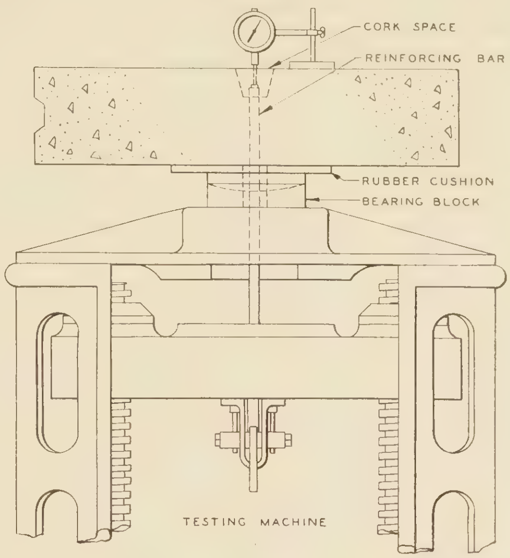


Figure 11.—Apparatus Used in Bond Tests of Bars Embedded Vertically

In the case of the bars placed horizontally a somewhat different scheme was adopted. The embedded end of each bar was drilled axially and tapped to receive a piece of ¼-inch drill rod 18 inches in length. This rod was sheathed with close-fitting tube of thin aluminum, which served to prevent any bond between the concrete and the drill-rod extension. The bars were inserted through horizontal holes in the ends of the slab forms at the mid-depth of the slab. The bar in bond extended into the concrete 6½ inches and the drill-rod extension continued entirely across the remainder of the slab section, the end being supported in a hole in the first transverse wooden separator. This served to hold the bar in position during the placing of the concrete and later, upon removal from the forms, to expose the extension to the embedded end of the reinforcing bar, permitting measurements of slip to be made.

Sections 1 and 8, illustrated in Figure 1, were both divided into two equal parts by a short longitudinal separator, which served to facilitate the handling of the bond specimens.

The tests were made in a universal testing machine, the specimens being supported on a large spherical bearing block provided with a radial hole which allowed the projecting bar to extend through the block. This block was mounted on the fixed head of the testing machine, through which the projecting end of the reinforcing bar passed to be engaged by the grips in the movable head. A rubber cushion was placed between the specimens and the bearing block. During the test the head of the machine was lowered, by power, at a rate of 0.05 inch per minute (idling rate), thus exerting

TABLE 11.—*Unit bond stress at initial slip, in pounds per square inch*

Slab No.	Aggregate	Method	Horizontal embedment: 1-inch bars, Test 1	Horizontal embedment: 1-inch bars, Test 2	Horizontal embedment: 1-inch bars, Average	Horizontal embedment: ½-inch bars, Test 1	Horizontal embedment: ½-inch bars, Test 2	Horizontal embedment: ½-inch bars, Average	Vertical embedment: 1-inch bars, Test 1	Vertical embedment: 1-inch bars, Test 2	Vertical embedment: 1-inch bars, Average	Vertical embedment: ½-inch bars, Test 1	Vertical embedment: ½-inch bars, Test 2	Vertical embedment: ½-inch bars, Average
		DEFORMED BARS												
23	Gravel	Hand placed	473	1,310	892	1,133	750	942	996	172	584	568	607	588
4	Gravel	Vibrolithic	640	1,132	886	730	688	709	1,462	1,082	1,272	872	1,145	1,008
7	Gravel	Electric tamper	1,190	1,426	1,308	463	255	359	920	1,229	1,074	714	568	641
10	Gravel	Air hammer	1,318	1,552	1,435	847	759	803	1,352	1,473	1,412	1,117	916	1,016
24	Crushed stone	Hand placed	634	396	515	1,020	954	987	610	--------	610	805	722	764
5	Crushed stone	Vibrolithic	1,155	1,369	1,262	478	618	548	1,657	1,141	1,399	901	1,270	1,086
8	Crushed stone	Electric tamper	1,562	1,845	1,704	1,165	787	976	1,720	1,888	1,804	922	778	850
11	Crushed stone	Air hammer	1,190	1,270	1,230	605	865	735	998	856	927	1,332	984	1,158
3	Haydite	Hand placed	542	457	500	385	255	320	212	402	307	354	1,039	696
6	Haydite	Vibrolithic	770	1,019	894	322	815	568	1,070	757	914	758	803	794
9	Haydite	Electric tamper	1,085	1,062	1,074	960	891	926	1,166	768	967	666	1,138	902
		PLAIN BARS												
12	Gravel	Hand placed	313	310	312	272	304	288	--------	214	214	550	422	486
15	Gravel	Vibrolithic	261	108	184	479	213	346	272	356	314	239	316	278
18	Gravel	Electric tamper	237	275	256	--------	151	151	296	361	328	574	680	627
21	Gravel	Air hammer	292	351	322	541	277	409	254	300	277	322	381	352
13	Crushed stone	Hand placed	165	82	124	280	196	238	228	280	254	568	470	519
16	Crushed stone	Vibrolithic	125	157	141	320	376	348	450	234	342	547	461	504
19	Crushed stone	Electric tamper	195	316	256	347	237	292	212	124	168	632	576	604
22	Crushed stone	Air hammer	343	230	286	419	391	405	552	375	464	560	601	580
14	Haydite	Hand placed	226	258	242	--------	225	225	251	--------	251	506	387	446
17	Haydite	Vibrolithic	210	297	254	272	251	262	290	291	290	435	455	445
25	Haydite	Electric tamper	404	431	418	306	311	308	416	324	370	450	570	510

tension in the bar and tending to withdraw it from the concrete.

The initial and progressive slip of the steel in the concrete was determined by a micrometer dial reading in ten-thousandths of an inch. This dial was supported by a heavy base which rested on the upper surface of the concrete specimen. The stem of the dial rested against the embedded end of the reinforcing bar, or its extension. Figures 10 and 11 show the way in which the bond specimens were set up for test.

The weighing beam of the testing machine was kept in balance as the load was applied and readings of load were taken at the point of initial slip, at the maximum resistance to slip, and at intervals of 0.01 inch slip up to the point where slippage was too rapid to record. As an aid to the accurate determination of the load at the instant of initial slip, a small electric light situated in front of the testing-machine operator was placed in a circuit which was broken by the first movement of the steel bar in the concrete. The operator was thus able to keep the machine balanced and still catch the load reading at the desired instant. It is believed that in all cases the first load reading was obtained before the bar had slipped one ten-thousandth of an inch.

The age of all of the specimens at the time of test was nine months, and during this period they were stored out of doors.

The load in pounds per square inch of bond area at which initial slip occurred is given for each test in Table 11, and these same data are shown graphically in Figure 12.

Figures 13, 14, and 15, show the load-slip curves for all tests with the plain bars. In the tests with the deformed bars, as soon as the bond was broken the bars began to exert a wedging action in the concrete, and failure occurred either by rupture of the bar in tension or by a splitting of the concrete block.

The initial unit bond resistance of the deformed bars is consistently much higher than that of the plain bars as indicated by the values in Table 11. The 1-inch plain bars developed, on the average, only 26 per cent of the resistance of the deformed bars of the same size. The ½-inch plain bars developed 50 per cent of the resistance of the ½-inch deformed bars.

The size of the bar appears to influence the unit bond resistance. In the case of the deformed bars the 1-inch size consistently shows higher bond values than the ½-inch size, while for the plain bars exactly the reverse is true. This is shown in the following summary:

	Lbs. per sq. in. bond stress
Deformed bars:	
1 inch, horizontal embedment	1,064
½ inch, horizontal embedment	716
1 inch, vertical embedment	1,044
½ inch, vertical embedment	864
Plain bars:	
1 inch, horizontal embedment	254
½ inch, horizontal embedment	308
1 inch, vertical embedment	304
½ inch, vertical embedment	486

The data indicate that, on the average, the bars placed in a vertical position developed about 15 per cent more bond resistance than those which were horizontal.

A study was made to determine whether or not the materials from which the concrete was made affected the bond strength. Classifying the data according to materials and averaging all values, the following values are obtained:

Unit bond resistance at initial slip

	Plain bars	Deformed bars
	Lbs. per sq. in.	*Lbs. per sq. in.*
Haydite	335	739
Gravel	321	933
Crushed stone	345	1,035

Thus it would appear that for the low resistance values developed by the plain bars the materials used in the concrete did not have an appreciable effect, but that for high bond-resistance values, such as were developed by the deformed bars, the aggregate does affect the unit bond resistance. It is noted that the variations in these data are in the same sense as the modulus of

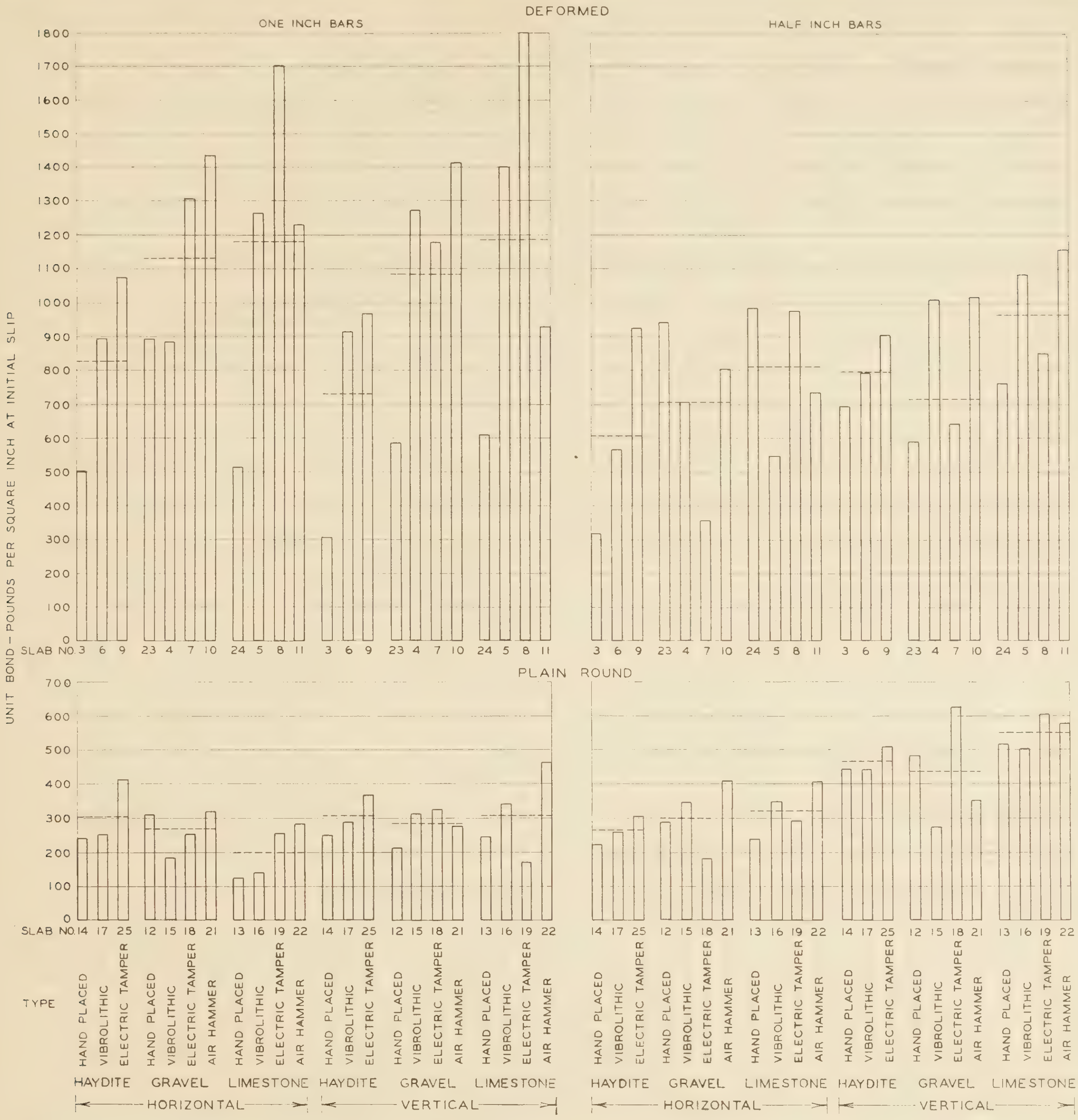


FIGURE 12.—MEAN UNIT BOND VALUES. DASH LINES REPRESENT AVERAGE VALUES FOR EACH GROUP

elasticity values for the three materials although considerably different in degree.

Passing now to a consideration of the effect of the various methods of placing used in this investigation on the initial unit bond resistance, we find that the bond strength of the deformed bars was very noticeably increased by the use of vibratory methods. In the case of plain bars the effect is much less marked. The following summary shows very clearly the effects just mentioned:

Unit bond resistance at initial slip

	Plain bars	Deformed bars
	Lbs. per sq. in.	*Lbs. per sq. in.*
Hand placed	300	642
Vibrolithic	309	945
Electric tamper	357	1,049
Air hammer	387	1,089

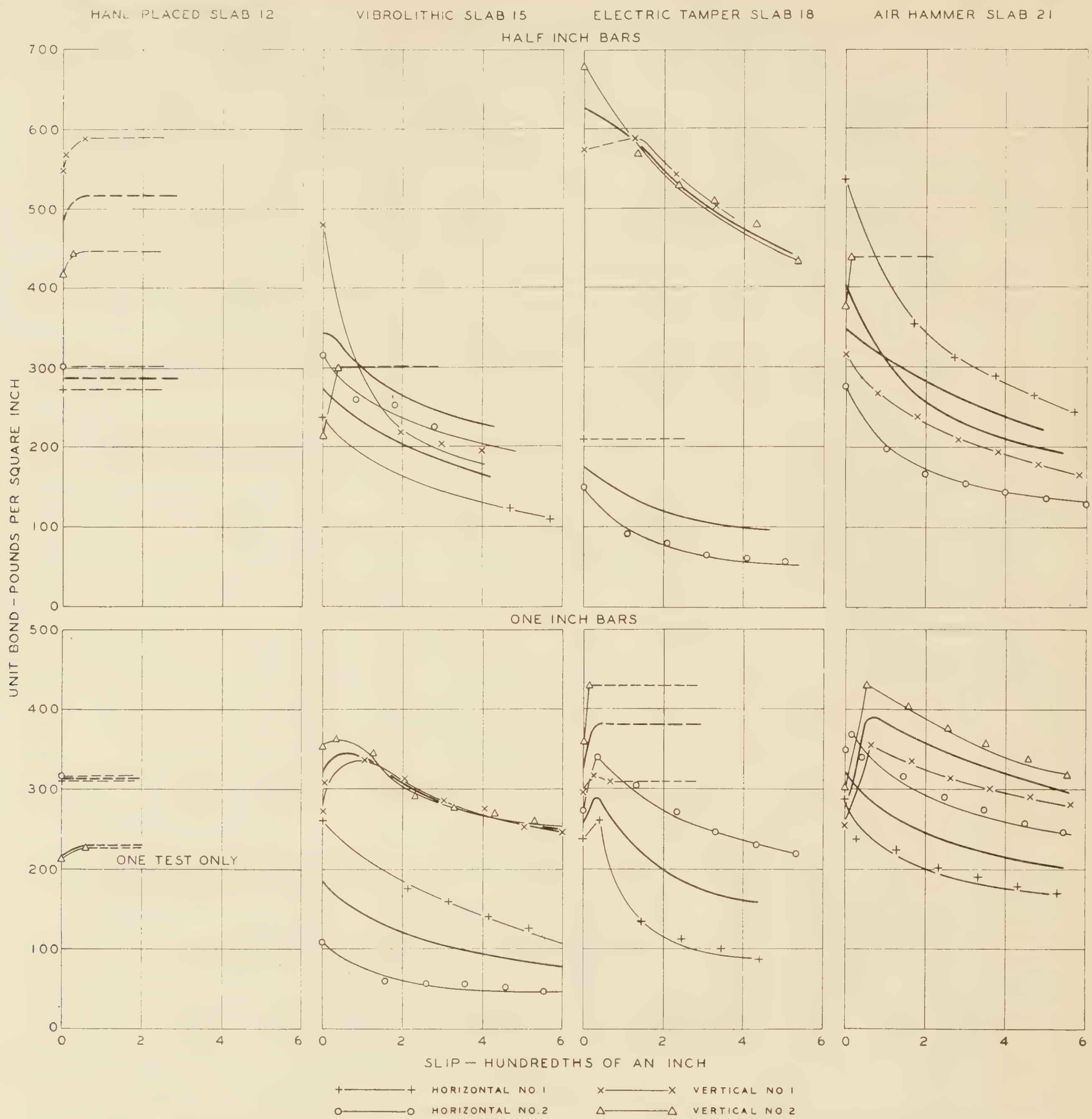


Figure 13.—Load-Slip Curves for Bond Tests of Plain Round Bars in Gravel Concrete

In the tests in which the plain bars were involved the indicated unit bond resistance at the initial slip very probably represents true bond between the steel and the concrete. With deformed bars, however, it is probable that the values measured are a combination of bond and shear.

The wedging action of the deformed bars was so marked that in about four out of every five tests the concrete block was split or the steel bar failed in tension. Of the ten 1-inch bars which were actually pulled out, six were from hand-placed concrete specimens and showed honeycomb around the bar. Of the seven ½-inch bars actually withdrawn, four were from hand-placed concrete showing honeycomb, and all seven of the bars were stressed beyond the elastic limit.

In all of the tests of plain bars the steel was withdrawn without exceeding the elastic limit of the material. It will be noted, in the load-slip curves for these specimens, that in many cases the maximum resistance did not develop at the initial slip but after a very slight movement had occurred. It is probable that this was true in all of the cases, but that the precision of measurement was not sufficient to have it show in the data.

These bond tests indicate that—

1. The apparent unit bond developed by deformed bars is consistently much higher than that developed by plain bars in the same concrete. This comparison is made at the loads at which the first slip was detectable with the apparatus used in these tests.

2. The size of the bar seems to influence the apparent unit bond. With plain bars the ½-inch size shows higher values than the 1-inch size, while with deformed bars the reverse is true.

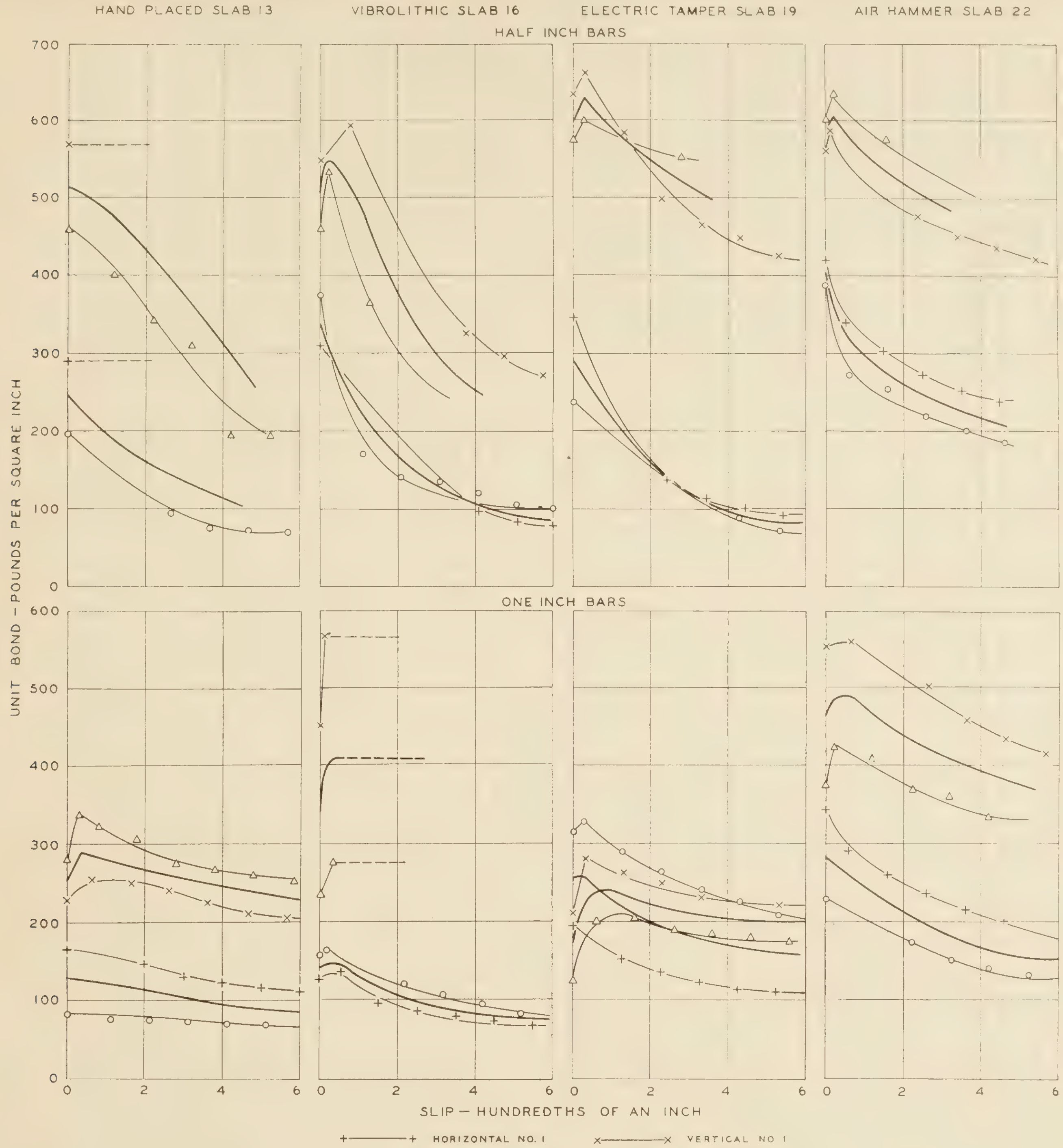


FIGURE 14.—LOAD-SLIP CURVES FOR BOND TESTS OF PLAIN ROUND BARS IN STONE CONCRETE

3. In general, the bars placed in a vertical position developed somewhat greater bond resistance than those placed horizontally.

4. Where slip occurs at comparatively low load, as is true of the plain bars, the aggregate in the concrete does not affect the resistance developed, but where deformed bars are employed the type of aggregate does influence the resistance to a considerable degree.

5. The method of placing used affects the bond resistance, the effect being more marked in the case of the deformed bars than in the case of plain bars. The vibratory methods of placing resulted in better bond than was obtained by the hand-placing method.

TESTS OF THE REINFORCED SECTIONS

As has been noted previously, two sections of each of the large slabs were reinforced with mats of steel bars of the same size and in the same amount as was being considered in connection with the design of floor slabs for the bridge. At the age of 6 months these sections were subjected to bending tests, observations of slab deflection and strain in the concrete being made. The details of the arrangement of the reinforcement in the concrete are shown in Figure 16.

The steel used consisted of ½-inch deformed bars of intermediate grade. The mean cross-sectional area of

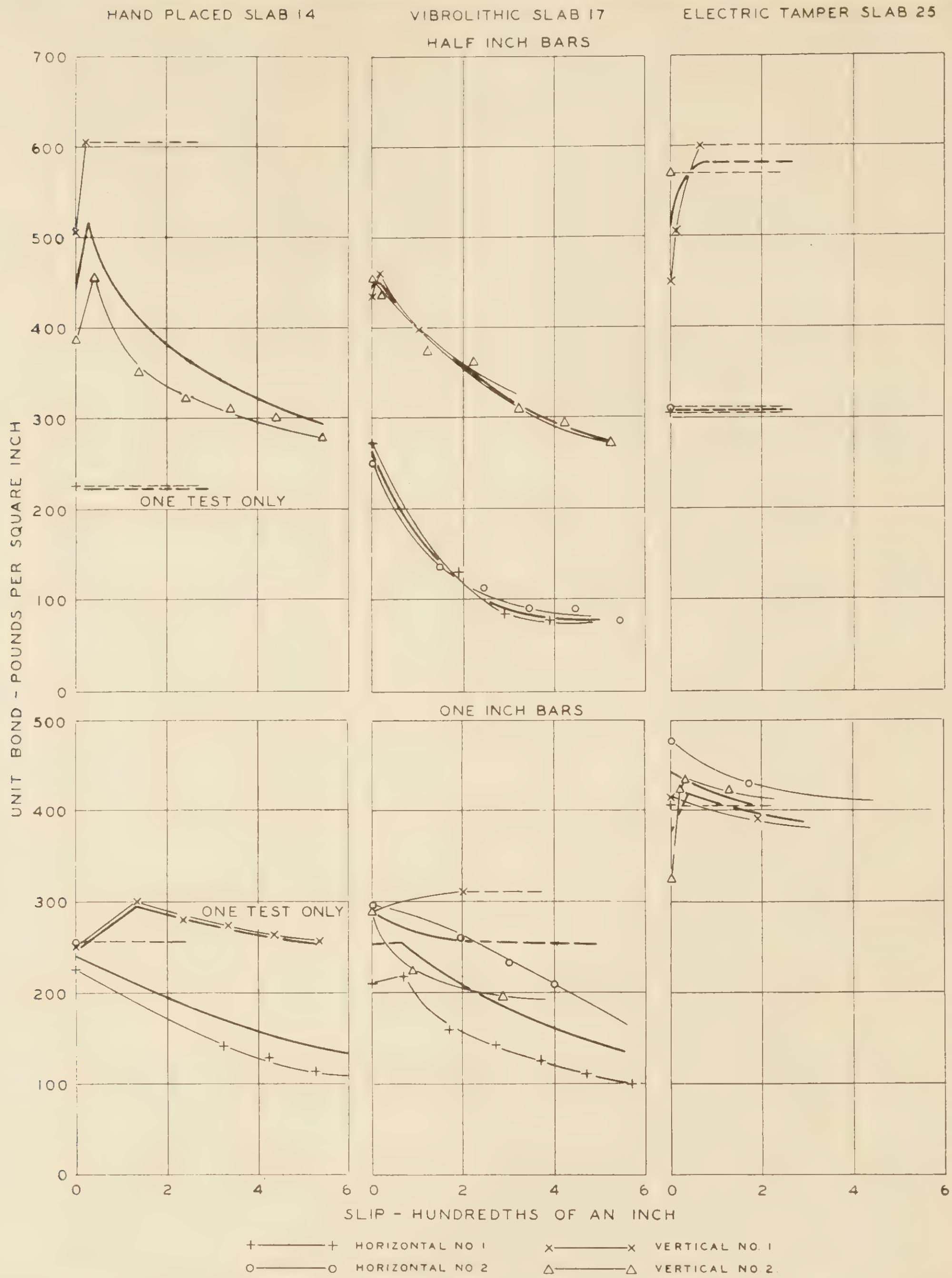


FIGURE 15.—LOAD-SLIP CURVES FOR BOND TESTS OF PLAIN ROUND BARS IN HAYDITE CONCRETE

these bars was measured and found to be 0.191 square inch. The elastic limit averaged 47,355 pounds per square inch (maximum 51,990, minimum 44,240) and the ultimate strength ranged from 84,400 to 72,040 pounds per square inch. The average percentage of elongation was 21.2 per cent. All of the specimens satisfactorily passed the cold bend test.

Figure 17 gives a general view of the way in which the reinforced sections were set up in the testing machine. The weighing table of the machine used is provided with wing extensions, and the slab supports rested on one of these. A 60-inch span was used and the load applied at the third points of this span by the framework shown in the figure just referred to. Horizontal freedom was provided at both load and reaction points by rollers acting against narrow steel plates embedded on the surface of the concrete. There were no particularly unusual features in the test set-up and the details will be omitted.

Deflection of the mid-point of the slab was measured with a micrometer dial reading directly in ten-thousandths of an inch and supported by a stiff tubular steel bridge provided with steel legs which rested on points set in the upper surface of the slab over the reaction supports. One of these legs was provided with a degree of horizontal freedom to take care of temperature and bending displacements. This apparatus is shown in position in the general view, Figure 17.

The deflection data were obtained on both sections from each of the 11 large slabs of the first series and the observations were made after each 5,000-pound increment of load had been applied. These data are given in detail in Table 12, and the load-deflection curves are shown in Figure 18.

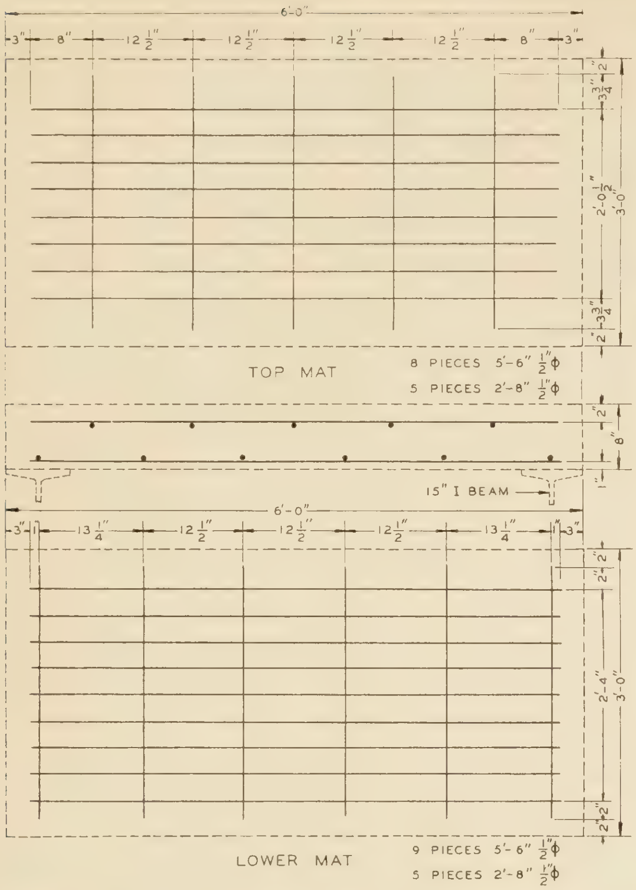


FIGURE 16.—PLACEMENT OF STEEL USED IN REINFORCED SLAB SECTIONS

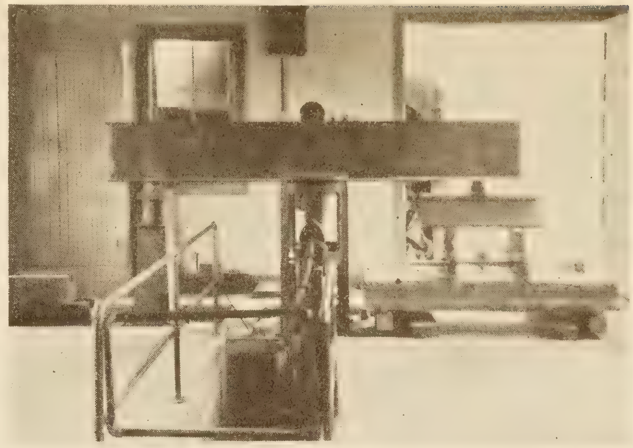


FIGURE 17.—GENERAL VIEW OF ARRANGEMENT FOR TESTS OF REINFORCED SLAB SECTIONS

TABLE 12.—*Detailed record of tests of reinforced concrete slab sections*

SLAB NO. 3, HAYDITE, HAND PLACED; SLAB THICKNESS=8¼ INCHES

Load in pounds	Deflection in inches				Stresses in section 3, in pounds per square inch				
	Section 3	Section 6	Mean	Corrected to 8-inch thickness	Gage No. 1	Gage No. 2	Gage No. 5	Mean	Corrected to 8-inch thickness
5,000	0.0045	0.0044	0.0044	0.0048					
10,000	.0093	.0091	.0092	.0101	210	240	246	232	276
15,000	.0142	.0151	.0146	.0160					
20,000	.0205	.0217	.0211	.0232	486	540	564	530	630
25,000	.0285	.0298	.0292	.0321					
30,000	.0433	.0488	.0460	.0505	1,026	1,026	1,056	1,036	1,234
35,000	.0588	.0648	.0618	.0680					
40,000	.0755	.0306	.0780	.0857	1,506	1,494	1,554	1,518	1,800
45,000	.0890	.0963	.0926	.1018					
50,000	.1127			.1238	1,986	1,980	2,058	2,008	2,390
Load at beam drop, pounds	54,895	50,000	52,448	50,800					

SLAB NO. 4, GRAVEL, VIBROLITHIC; SLAB THICKNESS=8½ INCHES

Load in pounds	Section 3	Section 6	Mean	Corrected to 8-inch thickness	Gage No. 1	Gage No. 2	Gage No. 5	Mean	Corrected to 8-inch thickness
5,000	0.0015	0.0019	0.0017	0.0021					
10,000	.0025	.0039	.0032	.0040	368	294	250	304	358
15,000	.0051	.0064	.0058	.0072					
20,000	.0067	.0098	.0082	.0102	647	603	544	598	680
25,000	.0121	.0144	.0132	.0165					
30,000	.0192	.0220	.0206	.0258	1,309	1,294	1,145	1,249	1,420
35,000	.0292	.0337	.0314	.0393					
40,000	.0424	.0497	.0460	.0575	2,176	2,205	1,983	2,121	2,410
45,000	.0554	.0625	.0590	.0738					
50,000	.0689	.0764	.0726	.0908	2,881	2,901	2,600	2,794	3,170
55,000	.0828								
Load at beam drop, pounds	59,875	54,390	57,132	53,600					

TABLE 12.—*Detailed record of tests of reinforced concrete slab sections*—Continued

SLAB NO. 5, LIMESTONE, VIBROLITHIC; SLAB THICKNESS=8⅝ INCHES

Load in pounds	Deflection in inches				Stresses in section 3, in pounds per square inch				
	Section 3	Section 6	Mean	Corrected to 8 inch thickness	Gage No. 1	Gage No. 2	Gage No. 5	Mean	Corrected to 8-inch thickness
5,000	0.0010	0.0015	0.0012	0.0016					
10,000	.0024	.0027	.0026	.0035	180	285	240	235	276
15,000	.0038	.0043	.0040	.0053					
20,000	.0053	.0061	.0057	.0076	390	570	450	470	552
25,000	.0071	.0087	.0079	.0105					
30,000	.0097	.0125	.0111	.0148	630	840	870	780	917
35,000	.0134	.0188	.0161	.0215					
40,000	.0238	.0308	.0273	.0364	1,365	1,505	1,530	1,467	1,730
45,000	.0331	.0462	.0396	.0528					
50,000	.0489	.0601	.0545	.0729					
55,000	.0637	.0748	.0692	.0923					
60,000	.0794	.0888	.0841	.1120					
Load at beam drop, pounds	63,125	63,520	63,322	58,500					

SLAB NO. 6, HAYDITE, VIBROLITHIC; SLAB THICKNESS=8½ INCHES

Load in pounds	Section 3	Section 6	Mean	Corrected to 8 inch thickness	Gage No. 1	Gage No. 2	Gage No. 5	Mean	Corrected to 8-inch thickness
5,000	0.0038	0.0037	0.0038	0.0046					
10,000	.0078	.0076	.0077	.0094	198	290	205	[illegible]31	340
15,000	.0113	.0122	.0118	.0144					
20,000	.0163	.0169	.0166	.0202	469	5[illegible]7	528	521	766
25,000	.0224	.0239	.0232	.0283					
30,000	.0328	.0339	.0334	.0407	924	1,022	931	959	1,390
35,000	.0466	.0508	.0487	.0594					
40,000	.0598	.0659	.0628	.0766	1,320	1,365	1,386	1,357	2,000
45,000	.0741	.0837	.0789	.0958					
50,000	.0948	.0997	.0972	.1185	1,789	1,966	2,046	1,934	2,840
Load at beam drop, pounds	54,385	53,975	54,180	47,950					

SLAB NO. 7, GRAVEL, ELECTRIC TAMPER; SLAB THICKNESS=8 INCHES [1]

Load in pounds	Section 3	Section 6	Mean	Corrected to 8 inch thickness	Gage No. 1	Gage No. 2	Gage No. 5	Mean	Corrected to 8-inch thickness
5,000	0.0025	0.0021	0.0023	0.0023					
10,000	.0052	.0048	.0050	.0050	273	230	301	268	268
15,000	.0087	.0085	.0086	.0086					
20,000	.0152	.0153	.0152	.0152	904	718	1,004	875	875
25,000	.0291	.0265	.0278	.0278					
30,000	.0498	.0421	.0460	.0460	1,995	1,880	2,439	2,105	2,105
35,000	.0688	.0568	.0628	.0628					
40,000	.0886	.0725	.0806	.0806	2,727	2,741	3,300	2,923	2,923
45,000	.1075	.0891	.0983	.0983					
50,000		.1058							
Load at beam drop, pounds	48,260	54,535	51,398	51,398					

SLAB NO. 8, LIMESTONE, ELECTRIC TAMPER; SLAB THICKNESS=8¼ INCHES

Load in pounds	Section 3	Section 6	Mean	Corrected to 8 inch thickness	Gage No. 1	Gage No. 2	Gage No. 5	Mean	Corrected to 8-inch thickness
5,000	0.0019	0.0019	0.0019	0.0021					
10,000	.0038	.0040	.0039	.0043	287	239	306	277	294
15,000	.0057	.0062	.0060	.0067					

[1] Lower steel displaced in section 3. One bottom bar out, mat raised ½-inch, moved 2 inches toward gages.

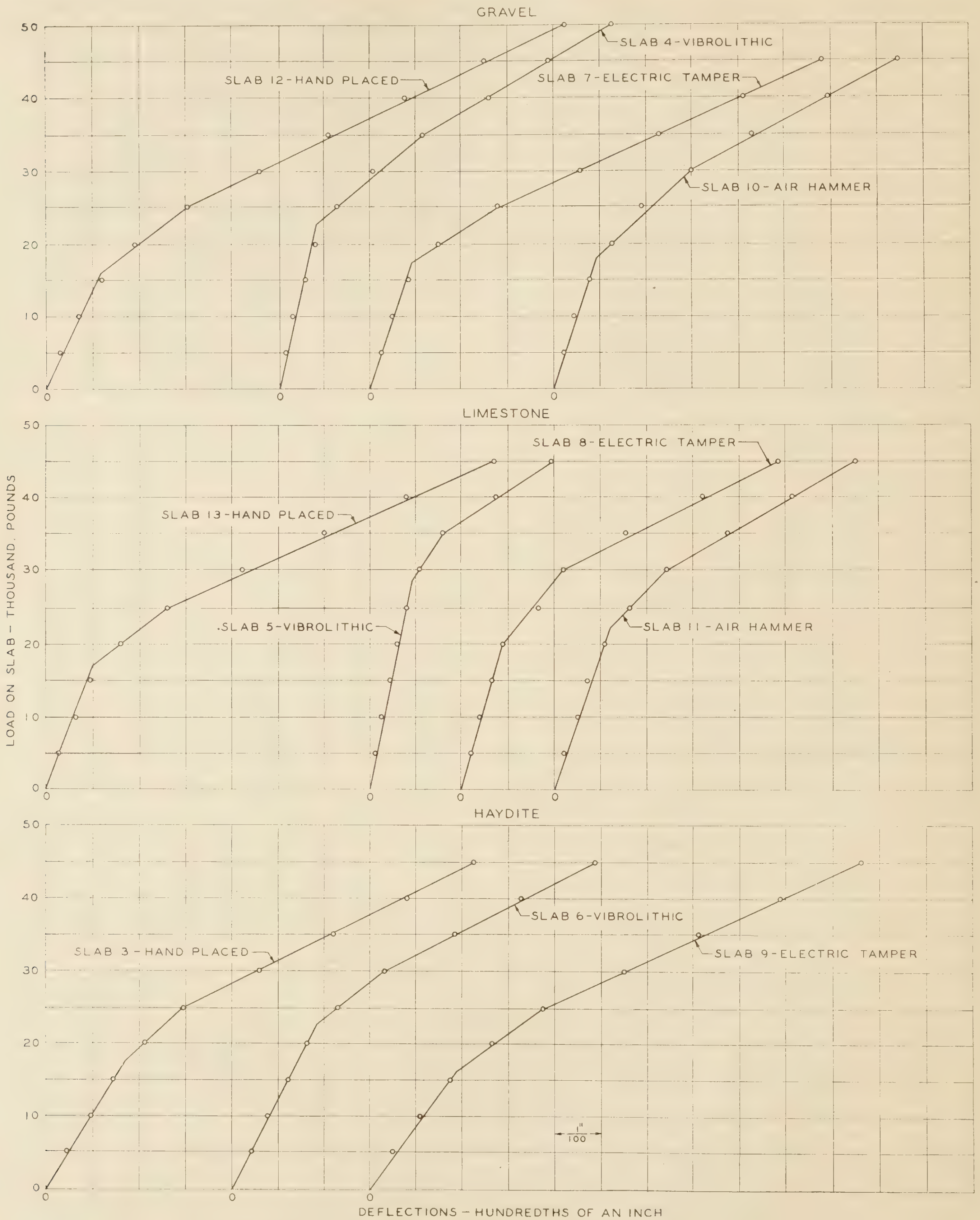


Figure 18.—Load-Deflection Curves for Reinforced Slab Sections

TABLE 12.—*Detailed record of tests of reinforced concrete slab sections*—Continued

SLAB NO. 8, LIMESTONE, ELECTRIC TAMPER; SLAB THICKNESS=8¼ INCHES—Continued

Load in pounds	Deflection in inches: Section 3	Section 6	Mean	Corrected to 8-inch thickness	Stresses in section 3, in pounds per square inch: Gage No. 1	Gage No. 2	Gage No. 5	Mean	Corrected to 8-inch thickness
20,000	.0076	.0092	.0084	.0093	734	545	753	677	720
25,000	.0112	.0167	.0140	.0155					
30,000	.0178	.0217	.0198	.0220	1,276	1,343	1,417	1,345	1,430
35,000	.0279	.0360	.0320	.0355					
40,000	.0418	.0519	.0468	.0520	2,233	2,300	2,185	2,239	2,380
45,000	.0552	.0679	.0616	.0684					
50,000	.0695	.0847	.0771	.0856	3,046	3,113	2,997	3,052	3,250
Load at beam drop, pounds	60,015	58,730	59,372	57,500					
SLAB NO. 9, HAYDITE, ELECTRIC TAMPER; SLAB THICKNESS=8 INCHES									
5,000	0.0050	0.0051	0.0050	0.0050					
10,000	.0107	.0110	.0108	.0108	306	282	252	280	280
15,000	.0165	.0190	.0178	.0178					
20,000	.0250	.0281	.0266	.0266	630	696	600	642	642
25,000	.0349	.0407	.0378	.0378					
30,000	.0516	.0587	.0552	.0552	1,114	1,236	1,092	1,147	1,147
35,000	.0661	.0760	.0710	.0710					
40,000	.0849	.0933	.0891	.0891	1,576	1,704	1,506	1,595	1,595
45,000	.1009	.1109	.1059	.1059					
Load at beam drop, pounds	49,410	48,730	49,070	49,070					
SLAB NO. 10, GRAVEL, AIR HAMMER; SLAB THICKNESS=8⅛ INCHES									
5,000	0.0021	0.0022	0.0022	0.0023					
10,000	.0045	.0051	.0048	.0050	307	256	256	273	281
15,000	.0075	.0086	.0080	.0084					
20,000	.0120	.0130	.0125	.0131	704	589	666	653	674
25,000	.0183	.0192	.0188	.0198					
30,000	.0297	.0301	.0299	.0315	1,395	1,421	1,332	1,383	1,426
35,000	.0442	.0436	.0439	.0462					
40,000	.0597	.0590	.0594	.0625	2,099	2,202	2,074	2,125	2,190
45,000	.0738	.0752	.0745	.0784					
50,000	.0885			.0931	2,829	2,919	2,740	2,829	2,920
55,000	.1040			.1095					
Load at beam drop, pounds	56,675	57,085	56,880	56,000					
SLAB NO. 11, LIMESTONE, AIR HAMMER; SLAB THICKNESS=8 INCHES									
5,000	0.0022	0.0024	0.0023	0.0023					
10,000	.0047	.0047	.0047	.0047	279	325	325	310	310
15,000	.0070	.0075	.0072	.0072					
20,000	.0110	.0112	.0111	.0111	759	743	743	748	748
25,000	.0151	.0173	.0162	.0162					
30,000	.0227	.0266	.0246	.0246	1,395	1,394	1,441	1,410	1,410
35,000	.0334	.0417	.0376	.0376					
40,000	.0464	.0559	.0512	.0512	2,309	2,355	2,371	2,345	2,345
45,000	.0587	.0712	.0650	.0650					
50,000	.0724	.0861	.0792	.0792	3,099	3,115	3,161	3,125	3,125
55,000	.0846	.1034	.0940	.0940					
Load at beam drop, pounds	60,590	58,420	59,505	59,500					
SLAB NO. 12, GRAVEL, HAND-PLACED; SLAB THICKNESS=8 INCHES									
5,000	0.0030	0.0030	0.0030	0.0030					
10,000	.0068	.0070	.0069	.0069	458	458	397	438	438
15,000	.0114	.0125	.0120	.0120					
20,000	.0172	.0214	.0193	.0193	1,038	1,038	901	992	992
25,000	.0269	.0343	.0306	.0306					
30,000	.0415	.0506	.0460	.0460	1,832	1,816	1,705	1,784	1,784
35,000	.0545	.0674	.0610	.0610					
40,000	.0701	.0844	.0772	.0772	2,626	2,564	2,583	2,591	2,591
45,000	.0868	.1030	.0949	.0949					
50,000	.1038	.1225	.1132	.1132	3,389	3,205	3,285	3,293	3,293
Load at beam drop, pounds	55,040	55,000	55,020	55,020					
SLAB NO. 13, LIMESTONE, HAND-PLACED; SLAB THICKNESS=8 INCHES									
5,000	0.0031	0.0028	0.0030	0.0030					
10,000	.0067	.0060	.0064	.0064	295	372	308	325	325
15,000	.0105	.0097	.0101	.0101					
20,000	.0169	.0151	.0160	.0160	796	898	886	860	860
25,000	.0283	.0238	.0260	.0260					
30,000	.0427	.0418	.0422	.0422	1,579	1,694	1,554	1,609	1,609
35,000	.0581	.0612	.0596	.0596					
40,000	.0731	.0827	.0779	.0779	2,208	2,375	2,260	2,281	2,281
45,000	.0873	.1061	.0967	.0967					
Load at beam drop, pounds	53,990	47,320	50,655	50,665					

Strain measurements were made in one of the two reinforced sections from each of the 11 large slabs of the first series and the observations were made after each 10,000-pound increment of load had been applied. Strains in the concrete were measured at five points in the upper surface of the slab, the location of which is shown in Figure 19. These measurements were made with a hand strain gage between brass points set 8 inches apart. Corrections were made for temperature changes in the gage. The data obtained are given in detail in Table 12 and shown graphically in Figure 20 where the average stress across the transverse axis of the slab is plotted as a function of the total load. The observed strains at the other two gage lengths along the longitudinal axis of the slab were used as a check on the distribution of stress in the middle third of the slab.

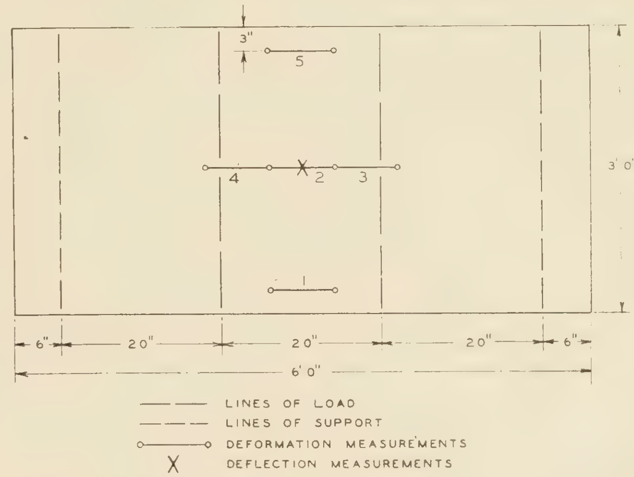


FIGURE 19.—GAGE LAYOUT ON REINFORCED SLAB SECTIONS

In converting the observed strains into the stress values given in Table 12, each value was multiplied by the modulus of elasticity of the concrete for that particular slab, as shown by the tests made at the 90-day age. These values of modulus of elasticity are very probably a few per cent lower at 90 days than they were at the time the reinforced sections were tested. Inspection of the data on the effect of age on the modulus of elasticity indicates that this difference would be small. (See Table 7.)

In Table 13 certain load and strength data for the reinforced slab sections have been assembled for purposes of comparison. The first two columns give the load on the slab at the time when the load-stress and and load-deflection curves respectively change in slope. It is realized that there are not a sufficient number of observations to fix this value very accurately, and the values given in the table were determined by projecting the initial and final slopes to their point of intersection. It is quite possible that the change in slope was gradual and that this change began at a lower load than is tabulated.

Column 3 contains the modulus of rupture of the concrete as determined by the tests of the sawed beams at the age of 6 months (the same age at test as the reinforced slabs). Columns 4, 5, and 6 contain stress values (in the concrete) arrived at in various ways. These values were computed in each case for the load A (Table 13) which is the load corresponding to the change in slope of the load-stress curve. Columns 4 and 5 contain stress values computed for the lower and upper surfaces of the concrete in the middle third of the slab. These values were

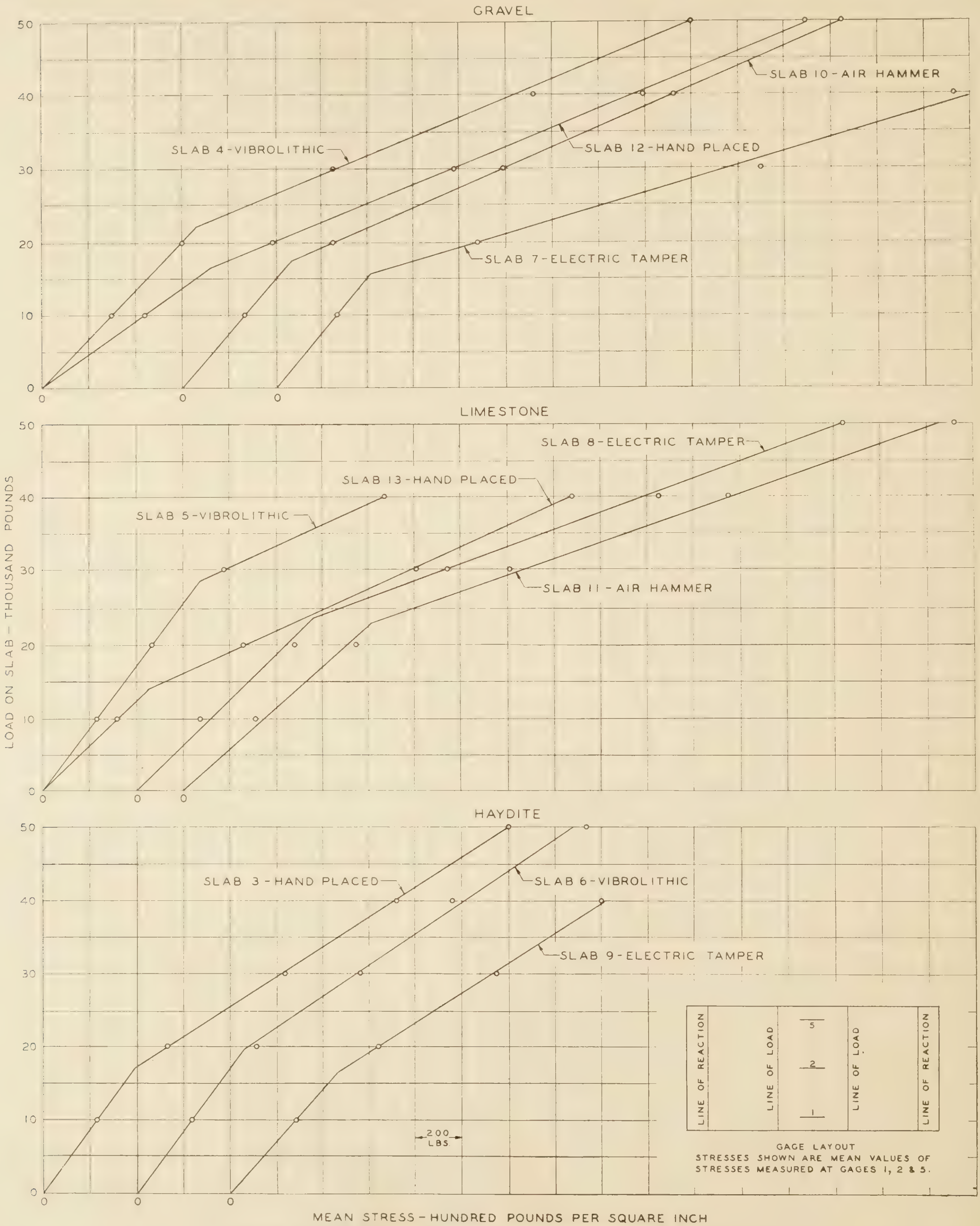


Figure 20.—Load-Stress Curves for Reinforced Slab Sections

Table 13.—*Comparative load and stress data for reinforced sections*

HAYDITE CONCRETE

Method of placing	Slab No.	Load in pounds at change of slope: Load-stress curve (load A)	Load in pounds at change of slope: Load-deflection curve	Modulus of rupture of sawed beams, in pounds per square inch, age 6 months	Computed stress in pounds per square inch for load A: Bottom of slab	Computed stress in pounds per square inch for load A: Top of slab	Computed stress in pounds per square inch for load A: Strain[1] × modulus of elasticity[2] $s=\delta E$
		(1)	(2)	(3)	(4)	(5)	(6)
Hand placed	3	17,500	18,000	496	486	485	410
Vibrolithic process	6	19,500	22,000	476	518	515	450
Electric tamper	9	16,250	16,500	433	458	462	450
LIMESTONE CONCRETE							
Hand placed	13	13,750	16,500	423	461	435	440
Vibrolithic process	5	28,250	27,500	710	817	803	660
Electric tamper	8	22,250	20,000	647	711	693	700
Air hammer	11	21,500	22,500	680	714	690	750
GRAVEL CONCRETE							
Hand placed	12	16,250	17,000	610	548	521	700
Vibrolithic process	4	22,000	22,500	636	662	643	660
Electric tamper	7	16,000	17,500	546	554	519	440
Air hammer	10	18,000	18,500	545	574	552	490

[1] Strain is the average of 3 gages on transverse axis of slab.
[2] Average modulus of elasticity from 90-day tests of cores from this same slab.

computed by the usual flexure formula, taking into account the actual dimensions of the slab and the amount and position of the longitudinal steel present in it. The ratio between the moduli of elasticity of the steel and of the concrete was computed on the basis of a value of 30,000,000 pounds per square inch for the steel and the value shown by the 90-day core tests for the concrete. Column 6 was computed from the measured strains in the concrete in the upper surface of the slab and the modulus of elasticity determined for the particular slab in question from the 90-day cores.

In Figure 21 a comparison is made of the relation between the various slabs as to—

(*a*) Load at which a change of slope occurs in the load-stress curve, and

(*b*) Modulus of rupture of the concrete at the same age.

A study of the data in Table 13 and Figure 21 clearly indicates that the change in slope of the load-stress curve is caused by the failure of the concrete in flexure. Considering the small number of tests which are represented by these data, it is believed that they are remarkably concordant on this point.

Since it is indicated that the first change in the structural behavior of the reinforced slabs occurred when the concrete failed in tension on the lower side of the slab, it will be of interest to examine the data to see what took place after the concrete ceased to bear its proportionate share of the tensile stress. The data show that after the first change in slope of the load-stress curve the structure continued to act elastically up to a certain point, which was indicated by the dropping of the weighing beam of the testing machine. In Table 14 the load on the various slabs at the time this elastic failure occurred are noted, together with the computed tensile stresses in the steel at these loads. In computing these values the dimensions of both the steel and concrete for the particular slab under consideration were used.

Figure 21.—Comparison of Values of Modulus of Rupture of Plain Concrete Beams and Values of Load on Reinforced Concrete Slabs at Initial Yield Point. In Left-hand Panel are Given Values of Load on Reinforced Slabs at Change in Slope of Load-Stress Curves, Corrected to 8-Inch Thickness. Values of Modulus of Rupture of Sawed Beams at 180 Days, Given in Right-hand Panel, are Means of 4 Tests on Gravel Concrete, and 8 Tests on Limestone and Haydite Concrete

Table 14.—*Loads on reinforced slabs at failure and corresponding stresses in steel*

Slab No.	Section No.	Aggregate	Method of placing	Total load	Tensile stress in steel
				Pounds	*Pounds per square inch*
3	3	Haydite	Hand placed	54,895	48,400
3	6	do	do	50,000	44,200
6	3	do	Vibrolithic process	54,385	46,000
6	6	do	do	53,975	45,600
9	3	do	Electric tamper	49,410	46,000
9	6	do	do	48,730	45,300
13	3	Limestone	Hand placed	53,990	48,200
13	6	do	do	47,320	44,000
5	3	do	Vibrolithic process	63,125	49,000
5	6	do	do	63,520	49,800
8	3	do	Electric tamper	60,015	50,000
8	6	do	do	58,730	50,000
11	3	do	Air hammer	60,590	52,200
11	6	do	do	58,420	50,500
12	3	Gravel	Hand placed	55,040	47,600
12	6	do	do	55,000	47,600
4	3	do	Vibrolithic process	59,875	48,500
4	6	do	do	54,390	44,100
7	3	do	Electric tamper	48,260	50,000
7	6	do	do	54,535	46,500
10	3	do	Air hammer	56,675	49,100
10	6	do	do	57,085	50,600

It is evident from this tabulation that the second change in structural behavior occurred as a result of the steel being stressed to the yield point. It is recalled that the yield point of the reinforcement was found to range from 44,240 to 51,990 pounds per square inch.

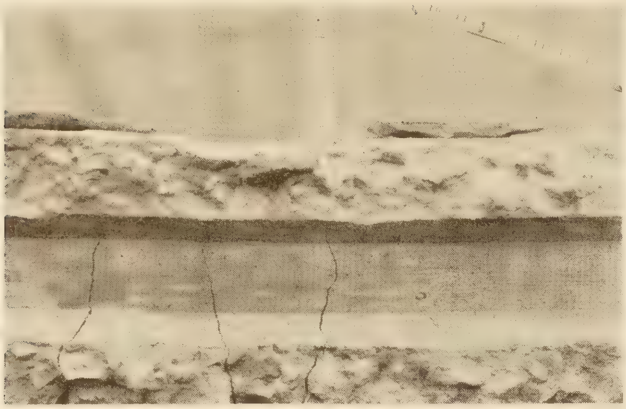


Figure 22.—Failure of a Reinforced Slab. Note Wide Cracks Remaining After Removal of Load and Spalling of Concrete on Compression Surface of Slab

Thus it may be concluded that the ultimate failure of the reinforced slabs was due to the failure of the steel in tension. The appearance of the slab specimens also indicated this to be true. Figure 22 shows the appearance of the side of one of these slabs after failure. This particular slab spalled at the upper surface, as shown in the photograph. This spalling occurred just before final failure.

ABSORPTION TESTS

An effort was made to determine the relative ability of the concrete from the various slabs to absorb water. For this purpose two specimens 8 inches square and approximately 18 inches in length were selected from the broken sawed-beam specimens from each of the large slabs. These specimens were air-dried in a sheet-iron building for several weeks, until all of them had ceased to change in weight. The concrete was from 10 to 12 months old at the time that this drying-out period was completed. When all of the specimens had reached a condition of constant weight in the dry air of the building, they were immersed in a tank of water (at air temperature). After periods of immersion of 1, 2, 8, 28, 56, and 140 days they were removed from the water; the surface moisture was blotted off with a damp cloth; and the specimens were weighed.

The progressive absorption of the various concretes is given in Table 15. Average values for four specimens are tabulated, and these data are also shown graphically in Figure 23. In this figure it will be noted that for each aggregate the methods of placing have been arranged from left to right in the order of ascending values.

From these data it would appear that the vibratory methods of placing produced a perceptibly less absorptive concrete than was obtained by hand placing. This difference was most marked with the concrete containing the Haydite aggregate and least with the concrete containing the gravel aggregate. Of the three vibratory methods, the Vibrolithic method of placing appears to be the most effective from the standpoint of reducing absorption. These absorption values appear to be, in a general way, a measure of the density of the concrete as indicated by the unit weight values given in Table 3. For each aggregate the highest absorption and lowest weight per cubic foot are found to relate to hand-placed concrete, while the lowest absorption and highest unit weight are found to be for concrete

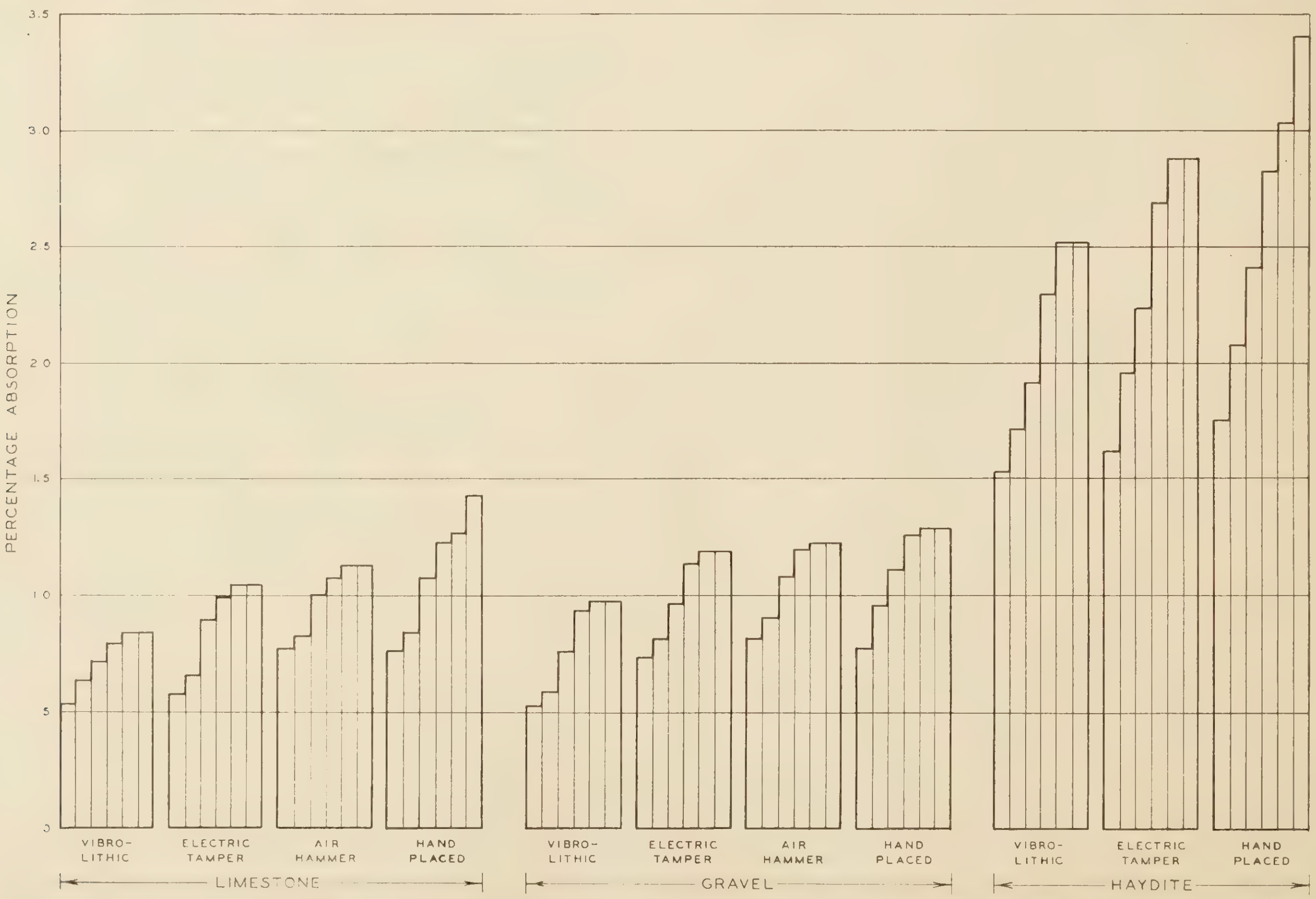


Figure 23.—Effects of Method of Placing and Type of Aggregate on Progressive Absorption of Concrete Specimens at 1, 2, 8, 28, 56, and 140 Days' Immersion. Each Value Given Is the Mean of Four Tests

TABLE 15.—*Percentage of absorption (based on dry weight) of 8 by 8 by 18 inch concrete specimens—Values given are averages of determinations on four specimens*

LIMESTONE AGGREGATE

Method of placing	Period of immersion (in days)					
	1	2	8	28	56	140
Vibrolithic process	0.53	0.64	0.72	0.80	0.84	0.84
Electric tamper	.58	.65	.89	1.00	1.05	1.05
Air hammer	.78	.83	1.00	1.08	1.13	1.13
Hand placed	.76	.84	1.07	1.23	1.27	1.43

GRAVEL AGGREGATE

Vibrolithic process	0.53	0.59	0.76	0.94	0.98	0.98
Electric tamper	.74	.82	.97	1.14	1.20	1.20
Air hammer	.81	.90	1.08	1.20	1.24	1.24
Hand placed	.78	.96	1.12	1.27	1.29	1.29

HAYDITE AGGREGATE

Vibrolithic process	1.53	1.71	1.94	2.30	2.51	2.51
Electric tamper	1.62	1.97	2.24	2.69	2.88	2.88
Hand placed	1.77	2.08	2.41	2.84	3.02	3.40

placed by the Vibrolithic method. The unit weight of the concrete containing gravel aggregate is about the same as that of the concrete containing the limestone and much higher than that containing Haydite, while from the standpoint of absorption the inverse relation is again found to be generally true.

RESISTANCE TO WEAR

The purpose for which these concrete slabs were to be used made the matter of resistance to surface wear of considerable interest. Tests were made on the 11 large slabs of the second series to determine the relative surface hardness or resistance to wear, using a special testing device developed by the bureau for this purpose. This apparatus is described in detail in a former issue of Public Roads[2] and consists of three narrow wheels of hardened steel set tangentially on a circular plate which is rotated about its center under a constant load at a constant speed of 35 revolutions per minute. The path followed by the wheels is a circle 21 inches in diameter.

These wheels follow each other around this path and soon begin to wear a groove. The rate at which the depth of this groove develops is determined by measurements with a micrometer dial after some definite number of revolutions of the plate and is taken as an indication of the relative surface hardness of the concrete.

As said before, this test was made on each of the 11 slabs of the second series. Two tests were made on each slab, and the age of the concrete at the time of test was 10 months.

The data obtained are shown graphically in Figure 24.

From these data it is evident that for all three aggregates the use of vibratory methods of placing tended to decrease the surface hardness of the concrete and that this effect was particularly pronounced in the case of the concrete containing the Haydite aggregate. The only exception is in the case of the concrete containing the gravel aggregate and placed by the Vibrolithic method. The concrete containing gravel aggregate which was placed by the two other vibrating methods consistently showed a softer surface than that which was

[2] Public Roads, vol. 10, No. 5, July, 1929, A Test for Indicating the Surface Hardness of Concrete Pavements.

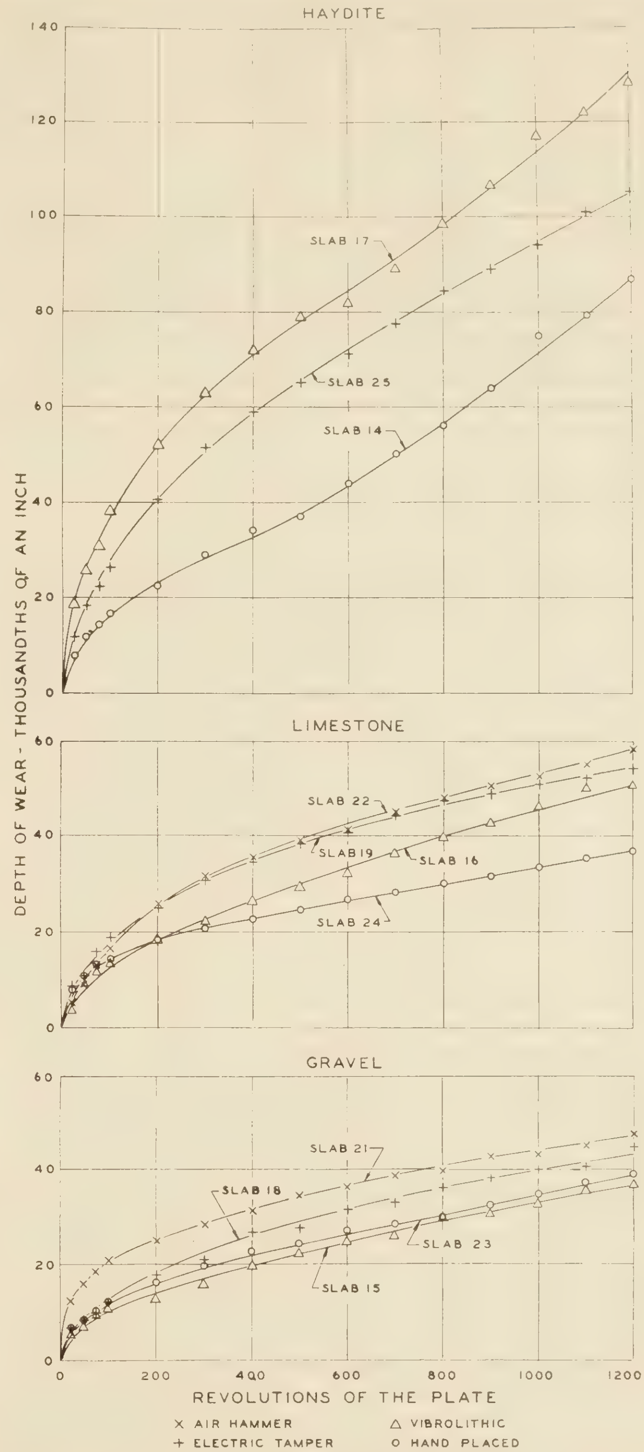
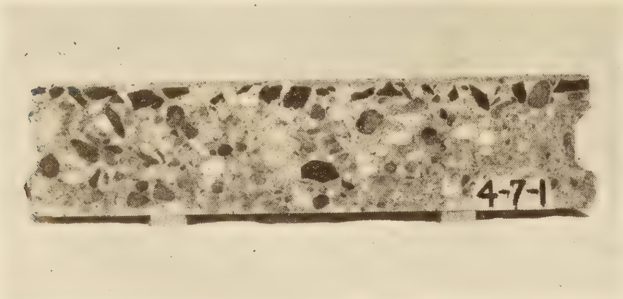


FIGURE 24.—RESULTS OF WEAR TESTS; AVERAGE CURVES DETERMINED BY TWO TESTS ON EACH SLAB

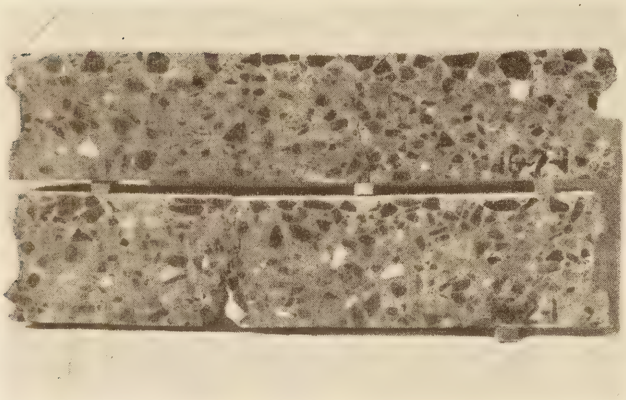
placed by the hand method. It is believed that this is due to the fact that the vibration tends to bring the water to the surface and this water carries with it the finer, less resistant particles. The result is that there is an excess of this material on or near the surface of the slab as the water evaporates.

Comparison may also be made between the different aggregates, and it will be noted that the depth of wear was approximately twice as great on the concrete containing Haydite as on the other aggregates.

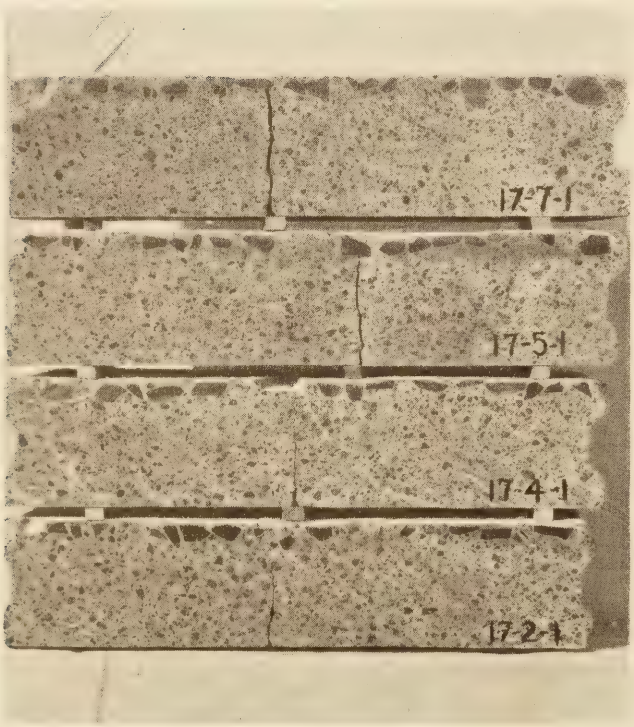
The addition of the trap rock to the surface of the slabs placed by the Vibrolithic process had for its purpose the protection of the surface from wear through the wear resistance of the stone. Presumably, when the mortar skin of the concrete becomes worn the layer of trap rock is exposed and assumes the burden of wear resistance.



GRAVEL AGGREGATE



LIMESTONE AGGREGATE



HAYDITE AGGREGATE

Figure 25.—Sawed Sections of Concrete Placed by Vibrolithic Process, Showing Final Position of Trap Rock Added During Finishing

The sawed beams furnished an unusual opportunity to study the position of these fragments of trap rock in the hardened concrete and to judge to what degree protection to the surface is afforded by them. Figure 25 shows typical faces of the sawed sections for all three types of aggregate. The pieces of stone in question can be identified without difficulty near the upper surface of all of the specimens. It is quite apparent that, so far as these slabs are concerned, the trap rock would be of little or no value as a surface wearing course.

RESISTANCE TO REPEATED FREEZING

Since durability is one of the most important qualities to be sought in exposed concrete work, an effort was made to obtain information regarding the relative resistance to weathering of the various concretes under test by subjecting specimens from each of the large slabs to alternate freezing and thawing.

Specimens 8 by 8 by 4 inches in size were sawed from portions of the sawed beams used in the flexure tests, one from section 2 and one from section 7 of each large slab, a total of 44 specimens. Four of the six faces of each specimen were sawed surfaces, one was the finished (or upper) slab surface, and the other was the surface left by the wood form on which the slab was cast. The specimens were separated into two groups of 22 each, one comprising the blocks cut from section 2 of each large slab, the other those from section 7. These two groups were alternated in the freezing cycle, one being frozen while the other was being thawed.

All of the specimens were subjected to 100 cycles of alternate freezing and thawing. Freezing was accomplished in a small compression refrigerator built especially for this work. The specimens were placed in the freezing chamber in a saturated condition and allowed to remain for 24 hours. Periodically thermograph records were made of the air temperature in the chamber and of the concrete temperature at the center of one of the specimens.

These charts showed that the minimum temperature of the concrete ranged from $-10°$ to $-15°$ C. over the 8-month period of the tests and that this minimum temperature was maintained for from 14 to 17 hours of each 24-hour cycle of freezing. These variations were caused by the variation in the temperature of the specimens when placed in the freezing chamber.

At the end of the 24-hour freezing period the specimens were removed from the refrigerator and immersed in a tank of water at room temperature, where they were allowed to remain for 24 hours.

All specimens were examined carefully at frequent intervals and any changes in appearance noted. The detailed observations for each specimen are given in Table 16, and photographs of all specimens are grouped in Figures 26, 27, and 28.

An examination of the specimens after the completion of 100 alternations of freezing and thawing leads to the conclusion that all of the specimens are in very good condition after a rather severe test. In general, the concrete is all sound and the sawed surfaces are practically intact. Differences in the resistance of the various groups of specimens can be detected, however, and these permit some comparison to be made of the effect on resistance of the aggregates and of the methods of placing. It will be noted that many of the observed effects of freezing listed in Table 16 are not of a serious nature so far as the mass of the concrete in the specimen is concerned.

Studying the data from the standpoint of the aggregates used, it is apparent that the concrete containing the siliceous gravel was affected least by the repeated freezings. This concrete was practically unchanged by the test in so far as could be determined by a careful examination of its physical appearance. The concrete containing the limestone coarse aggregate was more difficult to consolidate because of the angularity of the stone fragments. The water-cement ratio was higher than that of the concrete in which gravel was used and more void spaces are to be found around the coarse aggregate particles. The result was that freezing did considerably more damage to the specimens containing the limestone than to those containing the siliceous gravel. The void spaces proved to be particularly vulnerable to freezing attack.

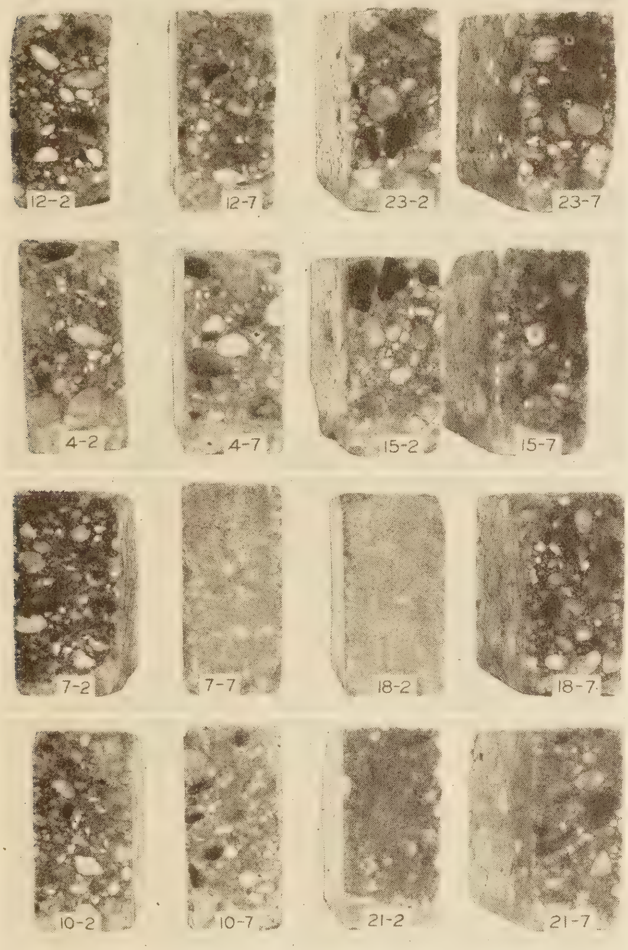


FIGURE 26.—RESISTANCE OF GRAVEL CONCRETE SPECIMENS TO REPEATED FREEZING. SLAB AND SECTION NUMBERS ARE GIVEN ON THE PHOTOGRAPHS. SLABS 12 AND 23, HAND PLACED; 4 AND 15, VIBROLITHIC; 7 AND 18, ELECTRIC TAMPER; 10 AND 21, AIR HAMMER

In only one instance was any disintegration of the limestone aggregate noted, the weakness apparently being in the mortar around the void spaces and on the finished surface which was originally the top of the slab. This indicates that the greater damage noted in the specimens containing the crushed-stone aggregate was possibly due to the fact that a higher water-cement ratio was required for workability rather than to the fact that limestone was used. All of the specimens containing the Haydite aggregate showed isolated

TABLE 16.—*Detailed observations of the effect of repeated freezing*

Type	Slab No.	Sec. No.	Effects observed
GRAVEL AGGREGATE			
Hand placed	12	2	All specimens unchanged after 100 alternations.
		7	
Do	23	2	
		7	
Vibrolithic process	4	2	Do.
		7	
Do	15	2	
		7	
Electric vibrator	7	2	None.
		7	Slight scaling on finished face (toward the front of the photograph) noted after 80 alternations.
Do	18	2	None.
		7	Do.
Air Hammer	10	2	Do.
		7	3 small pieces of coarse aggregate out of sawed surface after 80 alternations.
Do	21	2	None.
		7	Do.
LIMESTONE AGGREGATE			
Hand placed	13	2	Some crumbling of mortar toward a honeycombed area after 61 alternations.
		7	Some crumbling of mortar around a honeycombed area after 80 alternations.
Do	24	2	The lower third of this specimen was badly honeycombed, and after 59 alternations the mortar began to crumble. Considerable disintegration noted after 72 alternations.
		7	Some crumbling of mortar at 1 small spot on a sawed face noted after 80 alternations. No change after 100 alternations.
Vibrolithic process	5	2	Slight softening of the mortar above the trap rock on the finished surface apparent after 80 alternations.
		7	1 piece of limestone 1½ inches in diameter disintegrated after 72 alternations.
Do	16	2	Slight softening of finished surface noticeable after 80 alternations.
		7	Section of mortar on finished face, above the trap rock, loosened after 80 alternations.
Electric vibrator	8	2	Finished surface softened to a depth of 1/16 inch after 80 alternations.
		7	None
Do	19	2	Finished surface softened to a depth of 1/16 inch. First noted after 80 alternations (to the front of photograph).
		7	Finished surface softened to a depth of 1/16 inch. First noted after 80 alternations.
Air hammer	11	2	Finished surface showed scale after 61 alternations, and 1 corner, where honeycombed, crumbled after 80 alternations.
		7	Mortar on finished surface softened. Apparent after 72 alternations.
Do	22	2	Do.
		7	Mortar on 1 sawed face noticeably softened after 80 alternations. The same effect noted on 1 part of the finished surface.
HAYDITE AGGREGATE			
Hand placed	3	2	Slight scale on finished surface. First apparent after 80 alternations. Also certain particles of the coarse aggregate appeared to soften under repeated freezing. After 30 alternations it was noted that the nonporous particles, which apparently were not completely fused in manufacture, began to soften.
		7	Condition the same as section 2.
Do	14	2	Some crumbling of the mortar around a honeycombed area in the bottom. Slight scale appeared on the finished surface after 59 alternations.
		7	Pronounced scale on finished surface noticed after 30 alternations.
Vibrolithic process	6	2	The finished surface disintegrated to a depth of ⅛ to ¼ inch. One corner disintegrated where it was honeycombed. First noted after 80 alternations.
		7	The finished surface disintegrated to a depth of ¼ inch after 30 alternations. Some crumbling of the mortar noted after 80 alternations.
Do	17	2	Slight scale apparent on the finished surface after 80 alternations.
		7	Softening of the mortar noted at 2 small spots on the sawed surfaces after 30 alternations. The finished surface above the trap rock disintegrated to a depth of ⅛ to ¼ inch after 80 alternations.
Electric vibrator	9	2	The finished surface scaled to a depth of ¼ inch after 94 alternations.
		7	Slight scale noted on 1 sawed face after 30 alternations.
Do	25	2	No apparent effect. (One corner broken in handling.)
		7	Slight softening of 1 sawed face noticeable after 80 alternations.

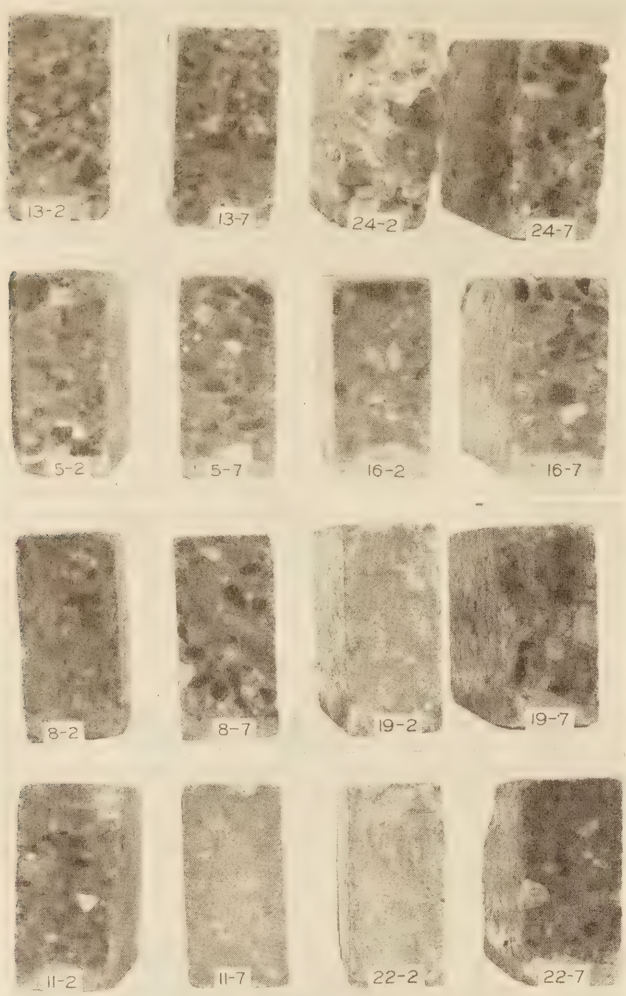


FIGURE 27.—RESISTANCE OF STONE CONCRETE SPECIMENS TO REPEATED FREEZING. SLAB AND SECTION NUMBERS ARE GIVEN ON THE PHOTOGRAPHS. SLABS 13 AND 24, HAND PLACED; 5 AND 16, VIBROLITHIC; 8 AND 19, ELECTRIC TAMPER; 11 AND 22, AIR HAMMER

particles of coarse aggregate which appeared to soften after repeated freezing. These particles were smooth, dark-colored, and noncellular in structure and had the appearance of having been incompletely fused during manufacture. The cellular particles were not affected by the repeated freezings. The mortar in the specimens containing the Haydite aggregates showed the effects of the test in practically all of the specimens. It is worthy of note, however, that the sawed surfaces which exposed the interior of the mass to the freezing action showed much less effect than did the finished surfaces.

It appears to be true of all of the specimens that the least resistant surface is that which was left by the finishing process on the upper surface of the slab.

The effect of the various methods of placing on the resistance to repeated freezing is limited, so far as these data indicate, to the effect that the respective methods have on the amount and condition of the mortar surface on the finished slab. The methods of placing which make use of vibration tend to bring fine material and water to the surface of the concrete. This was mentioned in the discussion of wear resistance as

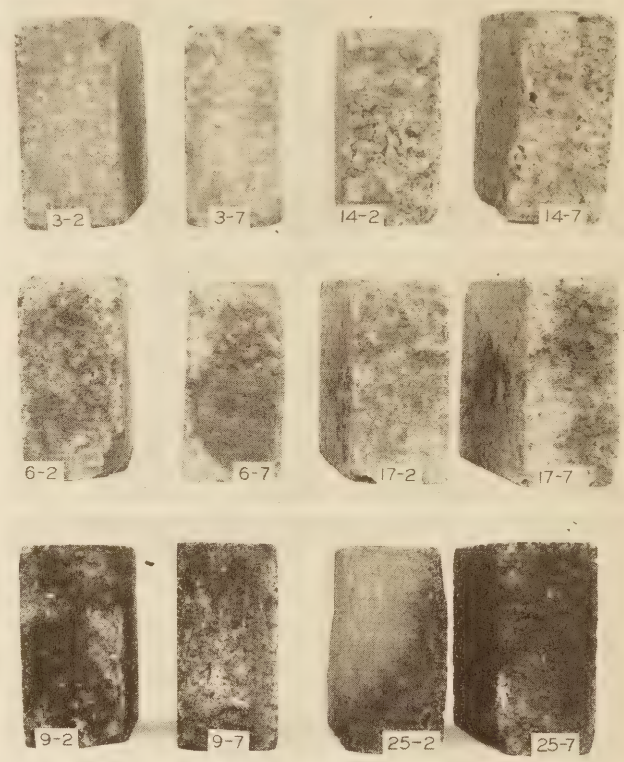


FIGURE 28.—RESISTANCE OF HAYDITE CONCRETE TO REPEATED FREEZING. SLAB AND SECTION NUMBERS ARE GIVEN ON THE PHOTOGRAPHS. SLABS 3 AND 14, HAND PLACED; 6 AND 17, VIBROLITHIC; 9 AND 25, ELECTRIC TAMPER

affecting the surface hardness of the concrete, and it is interesting to note that the surfaces left by the vibratory methods of placing were apparently less resistant to freezing than those obtained by the hand-placing method.

BULLETIN ISSUED ON ELECTRICAL EQUIPMENT FOR MOVABLE BRIDGES

A bulletin intended to assist bridge engineers in designing electrical equipment on movable bridges has recently been issued by the Bureau of Public Roads. The bulletin is designated as Technical Bulletin No. 265, Electrical Equipment on Movable Bridges, by Condé B. McCullough, Albin L. Gemeny, and W. R. Wickerham.

The first section of the bulletin deals with the fundamentals of direct-current and alternating-current motors and discusses the selection of motors for different services. The second section discusses the control and interlocking of operations, describing current practice in the regulation of motors, control of sequence of operations, and the use of various protective devices. This discussion is followed by a description of four different installations of electrical equipment, accompanied by diagrams showing the wiring and arrangement. The last portion of the bulletin deals with new developments in electrical bridge control, including variable-voltage control, speed-matching indicators for vertical-lift bridges, and light-sensitive relays.

Copies of this publication may be obtained from the Office of Information, United States Department of Agriculture, Washington, D. C.

HIGHWAY RESEARCH BOARD HOLDS ANNUAL MEETING

THE ELEVENTH annual meeting of the Highway Research Board was held on December 10 and 11 at the building of the National Academy of Sciences and the National Research Council, Washington, D. C. The session was opened at 10 a. m., Thursday, December 10, with an address of welcome by Dr. Albert L. Barrows, assistant secretary of the National Research Council, followed by an address by the chairman of the Highway Research Board, H. S. Mattimore, engineer of materials, Pennsylvania Highway Department. Mr. Mattimore presided at the meeting.

The Thursday morning session was devoted to the report of the committee on maintenance, B. C. Tiney, maintenance engineer, Michigan Highway Department, chairman; the report of a special investigation of dust-laying materials and methods, by Fred Burggraf, research engineer, Highway Research Board; and the report of the project committee on correlation of research in mineral aggregates, W. J. Emmons, director, Michigan State Highway Laboratory, chairman.

The afternoon meeting (C. M. Upham, of the American Road Builders' Association, presiding) was occupied with the following subjects: The report of the committee on materials and construction, H. S. Mattimore, chairman; the report of the project committee on curing of concrete pavement slabs, F. C. Lang, engineer of tests and inspection, Minnesota Highway Department, chairman; Compaction of Fills as Affected by Type and Size of Hauling and Other Equipment, by A. K. Haxtun, assistant engineer, American Road Builders' Association; Functions of Steel Reinforcement in Concrete Pavements and Bases, by C. A. Hogentogler, F. A. Robeson, and E. A. Willis, division of tests, United States Bureau of Public Roads; and the report of the special investigation of methods for evaluating field inspection of culverts, by R. W. Crum, director, Highway Research Board.

On Friday morning a short general session was held at which Mr. Mattimore presided. At this meeting the report of the committee on highway finance was delivered by Thomas H. MacDonald, chief, United States Bureau of Public Roads, chairman. The remainder of the day was occupied with two simultaneous sessions.

The session on highway design and allied subjects was held in the lecture room, E. F. Kelley, chief, division of tests, United States Bureau of Public Roads, presiding. This meeting was devoted to the report of the committee on highway design, A. T. Goldbeck, director, bureau of engineering, National Crushed Stone Association, chairman; Some Principles of Soil Surveying and Mapping, by D. P. Krynine, research associate in soil mechanics, Yale University; and the report of the special investigation of the use of rail steel reinforcement on highway construction, R. L. Morrison, chairman.

The session on traffic and transportation took place in the auditorium, with E. W. James, chief, division of highway transport, United States Bureau of Public Roads, in the chair. The following reports were considered: The report of the committee on traffic, G. E. Hamlin, superintendent of maintenance, Connecticut Highway Department, chairman; Field Methods for Measuring Tire Wear, by A. A. Anderson and H. B. Wright, Portland Cement Association; and the report of the committee on transportation, T. R. Agg, assistant dean of engineering, Iowa State College, chairman.

At 1 o'clock on Thursday, December 10, a luncheon for contact men was held in the library of the National Academy of Sciences. The annual highway research dinner was held at 7 in the evening of the same day, at the Willard Hotel.

Space does not permit an adequate account of the many excellent papers submitted to the Highway Research Board at this meeting. The following paragraphs contain brief summaries of a number of reports which are considered of particular interest to highway engineers.

MAINTENANCE

Maintenance Costs, by H. L. Bishop, chief, division of construction, United States Bureau of Public Roads, discussed progress on a study of maintenance costs on Federal-aid projects in all the States. Because of lack of uniformity in accounting methods, the Bureau of Public Roads has adopted a system of rating the maintenance on Federal-aid projects by weighing the factors of surface, drainage, shoulders, structures, roadside cleaning, and traffic service according to their relative importance. Other articles on maintenance included Maintenance of Expansion Joints and Cracks in Concrete Pavements, by W. H. Root, maintenance engineer, Iowa Highway Commission; Fillers and Bedding Courses for Brick Pavements, by J. S. Crandall, professor of highway engineering, University of Illinois; and Ice Removal from Pavements, by B. C. Tiney maintenance engineer, Michigan Highway Department.

MATERIALS AND CONSTRUCTION

H. S. Mattimore, engineer of materials, Pennsylvania Highway Department, reporting for the committee on materials and construction, submitted a paper on The Effect of Hot Cement on Time of Set and Concrete Quality, in which he discussed the effects produced by the use of cement which when received on the work retains a considerable part of the heat generated during manufacture. Occasional temperatures as high as 150° to 200° F. have been noted. In The Durability of Concrete as Affected by the Water-Cement Ratio, Mr. F. H. Jackson analyzed the evidence concerning the relation between resistance to frost action and the water-cement ratio, the maximum water-cement ratio allowable for concrete exposed to the weather, and the relation between strength and durability. A decided relation was found to exist between the water-cement ratio and durability. There was also a committee report on Volume Charges in Concrete, by C. H. Scholer, professor of applied mechanics, Kansas State College.

The paper on Functions of Steel Reinforcement in Concrete Pavements, by Messrs. Hogentogler, Robeson, and Willis pointed out the fact that investigations of steel reinforcement carried on by the Highway Research Board in 1924 and 1925 disclosed benefits furnished by reinforcement on concrete pavements not readily explainable by the conventional theory of reinforced concrete design. Subsequent study indicates that some of the benefits are due to the influence the reinforcement exerts upon the internal stresses and other factors which affect the strength and durability of concrete. The report analyzed the phenomena occurring during and subsequent to the hardening period and discusses the successive shrinkage, expansion, warping, and other distortions which may occur as the result of natural causes as well as traffic. The attempt was made to explain the function of steel reinforcement with respect to the stresses caused by these phenomena.

FINANCE

Mr. MacDonald's paper on highway finance dealt with the cooperative project being conducted by the Bureau of Public Roads and the University of Wisconsin on The Relationship Between Road and General Taxes in the Various States. The first work of the investigation was carried on in Wisconsin and was summarized in the report.

HIGHWAY DESIGN

The report of the committee on highway design included a symposium on The Economics of Low Cost Bridges: Concrete, by E. M. Fleming, Portland Cement Association; Timber, by J. F. Seiler, American Wood Preservers' Association; and Steel, by F. H. Frankland, American Institute of Steel Construction. Mr. Fleming's paper analyzed methods of reducing first cost by the use of simpler types of concrete structures, higher strength concrete, rigid frame structures, and continuous slab viaducts. After analyzing methods of calculating the total carrying cost of wooden bridges, Mr. Seiler cited experience to show that creosoted timber will last from 35 to 40 years, after which time a bridge would probably be retired from obsolescence. Mr. Frankland gave examples of recent economical designs of steel bridges using continuous girders and beams and steel-plate floors, as well as the use of steel piles for substructures. The life of steel structures was placed at not less than 100 years.

Load Limitations on Highways, by Albin L. Gemeny, senior structural engineer, United States Bureau of Public Roads, discussed the relation between load limitations on roads and those on bridges, and presented a discussion of load limitations based on the design loads of the American Association of State Highway Officials.

Field Experiments in Subgrade Drainage and Treatment, by F. H. Eno, research professor of highway engineering, Ohio State University, discussed an investigation of 42,500 feet of porous or treated subbases beneath paved roads and approximately 25,700 feet of untreated paved roads, the comparisons being made on the basis of crack ratios. Upon pavements with no center joint, slag subbases yielded no longitudinal cracks; sand gave a longitudinal crack ratio of 0.095; gravel, 0.2; a cement-clay mixture, 0.29, untreated sections, 0.6.

In A New Theory of Frost Heaving, A. C. Benkelman, research engineer, and F. R. Olmstead, research assistant, Michigan Highway Department, analyzed the factors which cause the formation of ice plates in soil, and noted the elimination of frost heaving at some 200 locations in Michigan last winter through provision for drainage.

The article by D. P. Krynine, on Principles of Soil Surveying and Mapping for Road Purposes, recommended that in addition to the soil grouping defined by the Bureau of Public Roads the soils be further characterized with respect to permeability and stability. For the former the author suggested a method for determining the moisture equivalent of the soil by centrifuging, and for the latter a new test for shearing resistance.

TRAFFIC AND TRANSPORTATION

Papers on the general subject of traffic included Traffic Capacity, by A. N. Johnson, dean of engineering, University of Maryland, a discussion of a cooperative project by the Bureau of Public Roads and the University of Maryland to determine the capacity of 2, 3, and 4 lane roads; Vehicle and Highway Mechanics, by Dr. H. C. Dickinson, chief, heat and power division, United States Bureau of Standards; Traffic Survey Procedure, by W. Graham Cole, policyholders service bureau, Metropolitan Life Insurance Co.; Law Observance and Enforcement, by Burton W. Marsh, traffic engineer, Philadelphia; Highway Education, by Stephen James, director of extension, Highway Education Board; State Control of Traffic, by W. A. Van Duzer, director of traffic, District of Columbia; and The Driver, by Sidney J. Williams, director, public safety division, National Safety Council.

The committee on transportation submitted three papers: Tractive Resistance Studies with a Gas-Electric Automobile, by Ray Paustian, Iowa State College; Tractive Resistance and Tractive Effort of Motor Vehicles, by H. B. Shaw, director, engineering experiment station, North Carolina State College; and Air Resistance of Motor Vehicles, by W. E. Lay, professor of mechanical engineering, University of Michigan. Mr. Paustian's paper discussed the equipment and preliminary results obtained in measurements of tractive resistance, wind resistance, and power consumption at speeds up to 65 miles per hour. Mr. Shaw gave methods for computing tractive resistance and tractive effort at different speeds and grades. Professor Lay reported on wind-tunnel experiments with simple body forms, using different methods of simulating a road surface. The data showing the percentage of improvement due to stream-lining proved to be fairly consistent for all methods tried.

SUBGRADE REPORTS REPRINTED IN BULLETIN FORM

The four reports on the subject of subgrades which were published in the June, July, September, and October issues of PUBLIC ROADS have been reprinted under the title, Reports on Subgrade Studies. The following articles are included: Subgrade Soil Constants, Their Significance and Their Application in Practice (Parts I, II, and III); The Soil Profile and the Subgrade Survey; Procedures for Testing Soils for the Determination of the Subgrade Soil Constants; Graphical Solution of the Data Furnished by the Hydrometer Method of Analysis.

Single copies of this publication may be obtained from the United States Bureau of Public Roads, Willard Building, Washington, D. C. In case more than one copy is desired, they may be purchased from the Superintendent of Documents, United States Government Printing Office, Washington, D. C., at 40 cents each.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

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

ANNUAL REPORTS

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

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

Report of the Chief of the Bureau of Public Roads, 1927.

Report of the Chief of the Bureau of Public Roads, 1928.

Report of the Chief of the Bureau of Public Roads, 1929.

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

DEPARTMENT BULLETINS

No. *136D. Highway Bonds. 20c.

*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.

*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.

*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c

*660D. Highway Cost Keeping. 10c.

1279D. Rural Highway Mileage, Income, and Expenditures 1921 and 1922.

1486D. Highway Bridge Location.

DEPARTMENT CIRCULAR

No. 331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETINS

No. 55T. Highway Bridge Surveys.

265T. Electrical Equipment on Movable Bridges.

SEPARATE REPRINT FROM THE YEARBOOK

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

MISCELLANEOUS CIRCULARS

No. 62MC. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.

*93MC. Direct Production Costs of Broken Stone. 25c.

109MC. Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails, Flood Relief, and Miscellaneous Matters.

MISCELLANEOUS PUBLICATION

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

TRANSPORTATION SURVEY REPORTS

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

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

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

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

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

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.

Vol. 5, No. 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. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

NOVEMBER 30, 1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
		$	$				$	$				$	
Alabama	2,387.8	1,777,150.13	864,457.19	80.7		80.7						5,345,824.08	Alabama
Arizona	1,075.9	4,078,388.78	2,910,116.84	192.0	64.7	256.7	215,102.75	161,950.85	20.0	19.8	39.8	1,743,928.64	Arizona
Arkansas	1,889.3	4,328,556.75	2,130,261.65	104.8	85.6	190.4	362,032.53	181,016.26	17.3		17.3	1,967,317.90	Arkansas
California	2,142.0	11,715,639.73	4,827,535.30	290.7	36.7	327.4	1,468,368.92	636,146.13	36.8	3.6	40.4	3,408,511.66	California
Colorado	1,452.9	6,241,669.15	3,177,185.41	268.5	76.7	345.2	722,154.01	323,854.09	8.6	4.5	13.1	2,078,778.86	Colorado
Connecticut	282.7	3,627,221.97	1,279,199.52	28.7		28.7						706,266.13	Connecticut
Delaware	361.3	206,946.00	103,473.00	4.6		4.6	108,131.00	54,065.50	9.7		9.7	516,622.43	Delaware
Florida	554.4	6,147,898.61	2,872,456.15	174.4		174.4						3,046,471.31	Florida
Georgia	2,978.2	7,001,386.58	3,171,723.08	168.5	157.8	326.3	884,530.69	439,999.25	34.8	21.0	55.8	2,366,149.47	Georgia
Idaho	1,431.9	2,379,675.26	1,396,198.93	130.9	90.2	221.1	98,447.53	58,388.61	5.0		5.0	1,479,979.03	Idaho
Illinois	2,541.7	22,745,509.64	10,498,709.14	680.5	28.7	709.2	4,656,663.95	2,093,162.55	145.4	4.7	150.1	3,700,407.94	Illinois
Indiana	1,726.3	8,738,423.39	4,331,011.54	251.1		251.1	1,706,186.78	812,731.06	61.6	12.5	74.1	2,548,193.74	Indiana
Iowa	3,348.6	1,220,570.06	390,906.47	20.5	22.3	42.8	274,243.43	119,960.00	5.6	5.4	11.0	2,768,854.32	Iowa
Kansas	3,630.4	3,582,250.77	1,660,993.87	172.8	23.1	195.9	39,928.88	16,029.02	3.3		3.3	3,076,169.68	Kansas
Kentucky	1,784.8	3,490,600.04	1,595,570.07	201.0	27.4	228.4	70,254.64	35,127.31	.5	4.0	4.5	2,029,039.79	Kentucky
Louisiana	1,484.0	7,114,787.76	3,317,899.49	168.6	11.7	180.3	2,567,840.55	135,581.56	1.0		1.0	1,644,886.88	Louisiana
Maine	683.6	3,061,521.21	1,317,172.11	53.9		53.9	37,546.34	14,086.00	.9		.9	1,285,660.42	Maine
Maryland	762.2	806,057.95	331,912.28	21.3		21.3						911,088.46	Maryland
Massachusetts	754.8	12,589,382.59	3,727,732.44	100.4		100.4	1,463,155.49	494,520.65	17.6		17.6	1,080,261.61	Massachusetts
Michigan	1,926.2	10,176,759.43	4,439,342.63	353.8	49.8	403.6	779,121.45	363,477.50	32.0	2.5	34.5	3,921,044.25	Michigan
Minnesota	4,208.3	2,777,438.57	1,153,825.98	28.4	118.4	146.8	3,378,002.59	1,139,450.17	87.4	67.8	155.2	1,800,857.50	Minnesota
Mississippi	1,808.1	3,540,448.12	1,749,977.28	167.8	61.3	229.1	415,088.00	184,234.59		17.3	17.3	5,515,254.55	Mississippi
Missouri	2,869.6	4,881,374.86	2,009,012.56	130.4	20.8	151.2	1,225,128.41	544,039.45	46.7		46.7	3,062,740.27	Missouri
Montana	2,494.0	6,523,994.08	3,655,972.26	480.9	40.4	521.3	622,013.88	348,436.03	70.0	12.4	82.4	3,180,014.93	Montana
Nebraska	4,097.5	7,145,232.06	3,363,577.57	248.1	94.9	343.0	25,454.82	12,727.40		18.4	18.4	2,592,325.16	Nebraska
Nevada	1,208.1	2,149,725.38	1,456,085.40	64.8	193.1	257.9						1,567,343.61	Nevada
New Hampshire	407.4	1,101,793.55	407,090.45	21.7	4.3	26.0						534,433.03	New Hampshire
New Jersey	573.7	6,777,396.56	2,435,375.14	67.6	.8	68.4						1,699,922.94	New Jersey
New Mexico	2,245.9	1,625,133.61	1,158,777.51	67.4	11.7	79.1	119,577.99	75,541.82	2.4		2.4	1,766,406.18	New Mexico
New York	2,996.5	26,025,020.00	9,958,297.50	515.0		515.0	1,590,000.00	639,750.00	35.8		35.8	5,307,148.59	New York
North Carolina	2,180.6	2,275,820.34	1,113,982.67	91.3	7.4	98.7	261,394.20	121,927.92	12.4	8.0	20.4	3,865,293.35	North Carolina
North Dakota	5,074.3	1,884,553.78	945,968.83	151.5	226.1	377.6	1,015,133.41	523,049.78	133.7	130.2	263.9	2,107,901.63	North Dakota
Ohio	2,728.8	10,627,774.18	3,786,966.77	182.8	38.3	221.1	882,244.81	364,545.30	13.8	.4	14.2	4,395,480.71	Ohio
Oklahoma	2,139.4	6,013,724.70	2,940,380.04	170.2	105.1	275.3	40,576.56	22,438.83	.1		.1	2,533,407.28	Oklahoma
Oregon	1,514.1	3,566,920.57	1,991,150.18	115.1	35.6	150.7						2,200,453.19	Oregon
Pennsylvania	2,931.4	6,364,129.44	2,942,184.65	126.4		126.4	15,163.78	7,040.14	.1		.1	5,151,715.52	Pennsylvania
Rhode Island	249.7	1,087,677.93	309,140.62	14.3		14.3						662,607.52	Rhode Island
South Carolina	1,959.4	3,531,650.70	1,550,984.08	76.8	67.9	144.7						1,389,774.57	South Carolina
South Dakota	3,961.7	3,509,204.53	2,041,080.79	234.8	189.5	424.3	11,839.81	6,574.64				1,613,069.03	South Dakota
Tennessee	1,666.6	526,796.35	263,398.15	10.2	4.3	14.5	892,398.31	429,604.69	30.3	10.7	41.0	3,799,566.22	Tennessee
Texas	7,480.2	14,305,362.68	6,669,606.28	694.4	188.9	883.3	2,039,485.25	884,509.59	95.1	43.4	138.5	7,806,467.39	Texas
Utah	1,194.4	1,109,093.76	682,133.30	68.8	13.9	82.7	50,994.68	37,736.05	5.7		5.7	1,526,514.50	Utah
Vermont	321.2	787,303.55	341,347.86	22.3		22.3						541,920.57	Vermont
Virginia	1,796.2	3,981,506.27	1,846,614.77	180.3	22.0	202.3	619,850.20	309,768.55	21.9	7.3	29.2	2,031,764.06	Virginia
Washington	1,160.7	2,562,758.42	1,213,165.33	81.7	6.5	88.2	425,552.72	228,800.00	10.5		10.5	1,911,634.96	Washington
West Virginia	855.7	2,713,732.03	1,214,862.30	72.9	10.5	83.4	97,329.39	40,854.05	.6		.6	1,264,883.70	West Virginia
Wisconsin	2,581.0	6,909,497.23	2,171,619.53	179.4	60.3	239.7	16,741.75	6,000.00	.4		.4	2,699,243.18	Wisconsin
Wyoming	2,030.5	2,098,323.95	1,220,908.44	163.4	71.6	235.0	142,062.76	75,614.83	17.5	11.8	29.3	1,296,072.81	Wyoming
Hawaii	59.4	1,094,751.98	481,061.26	31.6		31.6	82,324.99	22,869.00	1.5		1.5	2,044,877.80	Hawaii
TOTALS	97,993.4	257,528,574.98	115,428,404.61	7,928.0	2,268.3	10,196.3	29,422,047.25	11,965,598.17	986.0	405.7	1,391.7	121,494,541.35	TOTALS

[1]The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

U. S. GOVERNMENT PRINTING OFFICE: 1931